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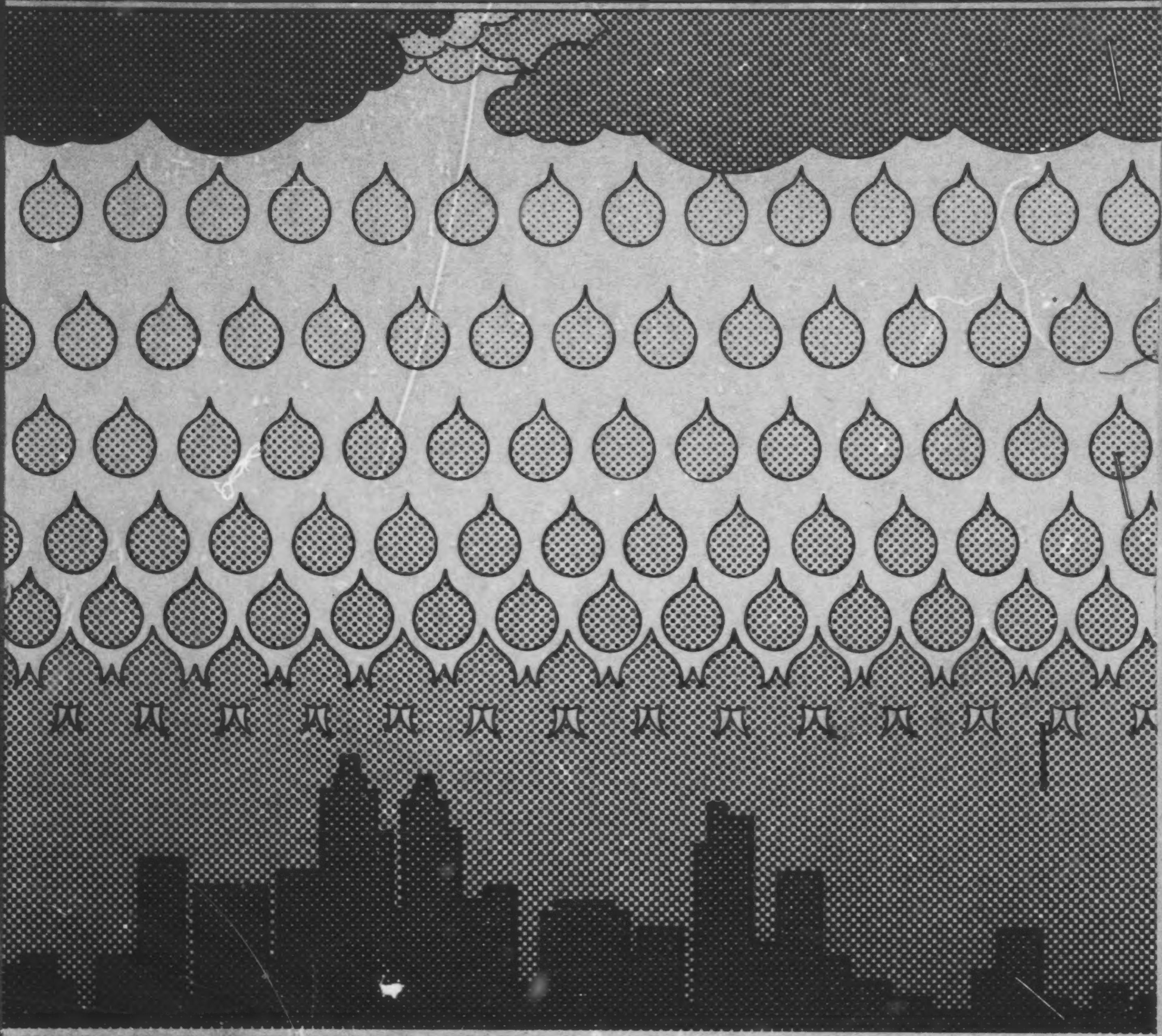

URBAN STORMWATER MANAGEMENT AND TECHNOLOGY
Update and Users' Guide

Municipal Environmental Research Laboratory
Office of Research and Development
U.S. Environmental Protection Agency
Cincinnati, Ohio 45268

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A continuation and reexamination of the state-of-the-art of storm and combined sewer overflow technology is presented. Essential areas of progress of the stormwater research and development program are keyed to the approach methodology and user assistance tools available, stormwater characterization, and evaluation of control measures. Results of the program are visible through current and ongoing master planning efforts. Assessment of urban runoff pollution is referenced to the developing national data base, localized through selective monitoring and analysis, and quantified as to potential source and magnitude using techniques ranging from simplified desktop procedures to complex simulation models. Stormwater pollutants are characterized by (1) source potential, (2) discharge characteristics, (3) residual products, and (4) receiving water impacts. Control and corrective measures are separated into nonstructural, termed Best Management Practices (BMPs), and structural alternatives. Best Management Practices focus on source abatement, whereas structural alternates roughly parallel conventional wastewater treatment practices of end-of-the-pipe correction. Structural alternatives may include storage (volume sensitive) and treatment (rate sensitive) options and balances. Multipurpose and integrated (dry-wet) facilities have been the most successful with process simplicity and operational control flexibilitiy prime considerations. Best Management Practices have decided benefits over structural alternatives--including lower cost, earlier results, and an improved and cleaner neighborhood environment--but lack quantified action-impact relationships. For combined sewer overflow abatement, increasing degrees of structural control is necessary. Successful program implementation is illustrated for several selected case histories.

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FOREWORD

The Environmental Protection Agency was created because of increasing public and government concern about the dangers of pollution to the health and welfare of the American people. Noxious air, foul water, and spoiled land are tragic testimony to the deterioration of our natural environment. The complexity of that environment and the interplay between its components require a concentrated and integrated attack on the problem.

Research and development is that necessary first step in problem solution and it involves defining the problem, measuring its impact, and searching for solutions. The Municipal Environmental Research Laboratory develops new and improved technology and systems for the prevention, treatment, and management of wastewater and solid and hazardous waste pollutant discharges from municipal and community sources, for the preservation and treatment of public drinking water supplies and to minimize the adverse economic, social, health, and aesthetic effects of pollution. This publication is one of the products of that research; a most vital communications link between the researcher and the user community.

As the base of information continues to expand through continuing research and development, it becomes increasingly important to transfer this knowledge through a concise compendium of urban stormwater practices. This report presents the most recent developments in the state-of-the-art of completed and ongoing storm and combined sewer management and abatement technologies.

Francis T. Mayo
Director
Municipal Environmental
Research Laboratory

ABSTRACT

A continuation and reexamination of the state-of-the-art of storm and combined sewer overflow technology is presented. Essential areas of progress of the stormwater research and development program are keyed to the approach methodology and user assistance tools available, stormwater characterization, and evaluation of control measures. Results of the program are visible through current and ongoing master planning efforts.

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Successful program implementation is illustrated for several selected case histories.

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The Republic of Technology is a world of obsolescence. Our characteristic printed matter is not a deathless literary work but today's newspaper that makes yesterday's newspaper worthless...

Bicentennial Essay
TIME, January 17, 1977

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SECTION 1

INTRODUCTION

The quality of the surface waters of the nation reflects the aggregate effect of all discharges. As objective levels of nondegradation and restoration rise and as broad strides of countermeasure implementation are achieved, the role of the heretofore "lesser" discharges becomes increasingly important.

In response to an exploding environmental consciousness on the part of its citizens, the United States has set forth the following goals through PL 92-500, the Federal Water Pollution Control Act Amendments of 1972:

1. "To restore and maintain the chemical, physical and biological integrity of the Nation's waters." [Section 101(a)].

2. "Wherever attainable, an interim goal of water quality which provides for the protection and propagation of fish, shellfish, and wildlife and provides for recreation in and on the water be achieved by July 1, 1983." [Section 101(a)].

These goals cannot be achieved without recognition and assessment of all source loads and the formulation and implementation of mitigation programs drawn from an equally broad base.

URBAN STORMWATER MANAGEMENT

Urban stormwater management programs address water pollution initiated by rainfall (or frozen precipitation) impacting on developed and developing areas. Pollution is intensified as particulates are scrubbed from the air; washed from the land, pavement, and building surfaces; scoured from the collection network; and finally resuspended, transported, and deposited within the receiving waters themselves.

When stormwater runoff and municipal wastewaters are intentionally carried in the same collector system, the spills (untreated discharges) are termed combined sewer overflow. Significantly, 56% of the population in the nation's cities with 100 000 or more inhabitants [1] are served by such combined or partially combined systems. When stormwater and municipal wastewaters are collected separately, cross-connections (either direct or indirect) frequently have been found, resulting in similarly degraded overflow qualities. Finally, the separately-collected or free-discharging stormwaters alone can produce mass releases of contaminants harmful to receiving waters and in violation of objective criteria.

Assessment

Surface runoff generated problems and appropriate mitigation measures are difficult to assess because:

- The events are irregular and unpredictable
- The impacts are likely to be highly time and location variable
- Other discharges or conditions tend to mask actual results
- Relatively little usable local data are available and new data are extremely time consuming and costly to obtain
- Mitigation measures are largely conceptual and effectiveness is ill defined

These difficulties and the unquestioned need for solutions have spawned over the past decade a major research and development effort both in the United States and in other nations around the world. The result has evolved in the development and application of a new technology which emphasizes time and spatial effects and total system consciousness. Solutions are found not only in improved hardware and process operations, but even more so in the stressing of management practices that limit the spread of the problem and attack it at its source.

Because the flow quantities are high, reaching one to two orders of magnitude greater than dry-weather flows, control--whether through flow balancing, multiple uses of facilities, runoff retardation, or combinations thereof--is the focus of cost-effective planning.

Planning Process

PL 92-500 contains complex and far reaching pollution control incentives and commits the federal government to eliminate pollution of the nation's waterways. Because of their impacts on funding and program guidance, three sections of the Act have major significance: Section 303(e), the State Continuing Planning Process; Section 208, Areawide Waste Treatment Management Planning; and Section 201, Facilities Planning.

Ideally, the 303(e) plan establishes the waste loads; the 208 plan provides the regional overview and designates the 201 area and the implementing agency; and the 201 plan develops a specific project that is the most environmentally sound and cost effective for achieving the stated water quality standards.

With respect to combined sewer overflows and stormwater discharges, present construction grant policy is [2]:

> Projects involving treatment and control of combined sewer overflows and stormwater discharges may be considered only after the planning process has clearly established their cost-effectiveness. Such projects must be

considered on a case-by-case basis after a careful review of all alternative control techniques has shown that, even after industrial effluent limitations and a minimum of secondary treatment for dry-weather municipal flows are achieved, the selected alternative is needed to protect the beneficial use of the receiving waters. See PRM 75-34 (PG-61).

In spite of the rigorous restrictions, a number of major combined sewer overflow abatement projects are being funded today. Selected milestone projects are described in Section 8 of this report.

Needs

Urban stormwater management is, in itself, a continuous process. Essential to its success is a constant process of innovation, demonstration, assessment, implementation guidance, and active program feedback. Eventual program costs will be in the $10 billions [3, 4, 5]. Such a program must be founded on proven capabilities, comparable methodologies and assessment criteria, an expanding data base, and a continuous effective technology transfer.

The difficulties cannot be regarded lightly. Much has been accomplished and clear benefits derived; however, the greatest challenge--the transition from planning to realization (with noteworthy exceptions)--is still before us.

State-of-the-Art Report

In 1972, the United States Environmental Protection Agency (EPA) through its Storm and Combined Sewer Section (Edison, N.J.) authorized the preparation of a comprehensive investigation and assessment of promising, completed, and ongoing urban stormwater projects, representative of the state-of-the-art in abatement theory and technology. The report, completed by Metcalf & Eddy, Inc., in December 1973, presented in textbook format a compendium of project information on management and technology alternatives within a framework of problem identification, evaluation procedures, and program assessment and selection.

In the process, over 140 projects, totalling over $90 million, awarded under the EPA Storm and Combined Sewer Technology Research, Development and Demonstration Program were reviewed, as well as other national and local milestone programs. The report, URBAN STORMWATER MANAGEMENT AND TECHNOLOGY: An Assessment, EPA-670/2-74-040, December 1974, is available through the National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22151 [Order number: NTIS-PB 240 687] and the Storm and Combined Sewer Section, EPA, Edison, N.J. 08817.

The objective of this project is to improve and accelerate the transfer of new technology in the field of urban stormwater management from the researcher to the potential user.

Presented as an UPDATE AND USERS' GUIDE, the report supplements the earlier work by directing attention to the latest developments in the field, through expansion of the data base, by the addition of example problems, and by reconstruction of key projects in a form more useful to potential decision makers. The UPDATE is designed to be used in conjunction with, and not as a replacement of, the earlier report.

Descriptions, methodologies, and problem solutions presume a general understanding on the part of the reader of urban stormwater problems and solution alternatives, such as could be gained from the earlier work or comparable firsthand experience. In this manner, it is hoped that redundancies are reduced and that new work and information are emphasized. Selected tabular information, particularly characterization data, costs, and performance criteria have been repeated where block comparisons are considered beneficial and where significant modifications have been made.

FORMAT

The report presentation is organized into five parts, each containing illustrative problem sets where applicable. A glossary of key terms is located in the appendix.

The first part, Approach Methodology, identifies the major planning guidance documents and tools available; highlights their utility in quantifying problems and setting up approaches; and demonstrates their applicability in program development.

The second part, Data Base and Normalization, provides an update of field data and approaches used to normalize these data for transferability and impact analysis.

The third part, Best Management Practices for Nonstructural Stormwater Control, summarizes recent progress in legislative, source, and nonstructural controls and attempts to assess their relative cost-effectiveness.

The fourth part, Unit Processes, provides a similar update on applied unit processes with emphasis on performance characteristics, flexibility, operational considerations, and potential cost effectiveness.

The fifth part, System Applications, emphasizes the total systems approach and illustrates through case history examples their development from concept to implementation and, if operational, to assessment.

References

The source material covered includes that which was released, published, or obtained through direct contact over the period from January 1974 through September 1976. Both United States and selected foreign literature are included. A complete bibliography is appended to the report.

Over 360 references were reviewed covering ongoing, new, and complete projects in the field of stormwater management. Considering the 33 month search span, this represents an average document generation of better than 10 per month which is indicative of the intense activity--and rapid obsolescence--of the technology and data base. Each reference reviewed was cataloged into one or more of seven broad categories and multiple subcategories as listed in Table 1. A breakdown, illustrating the distribution of source material across these categories, is shown in Figure 1.

TABLE 1. REFERENCE DISAGGREGATION AND RETRIEVAL

Category	Detailed description
1. Storage/treatment processes	• Inline storage • Offline storage • Swirl concentrators/regulators • Screening/microstraining • Sedimentation • Dissolved air flotation • Stabilization basin • Disinfection • Filtration/hyperfiltration • Biological treatment • Chemical treatment
2. Pollutant characterization	• Characterization values • Sludge/solids • Biological/microbiological • Chemical constituents • Street/land contaminants • Sediment • Nutrients • Heavy metals
3. Mathematical models	• Management • Costs • Storage and treatment • Rainfall/runoff • Collection/transport • Receiving water • Comprehensive/planning
4. Management planning	• Sediment control • Treatment control • System control • Economics • Comparison of alternatives • Source control • Sampling/data aquisition • Sewer separation • Problem characterization • Systems analysis
5. Case studies	• Pilot plants • Full-scale plant • Bench-scale tests • Full-scale systems
6. Miscellaneous articles	• Abstracts and bibliographies • Seminar papers • Cost information • O&M information • R&D projects • Legislation
7. Project memos	• Progress reports

DATA BANK

Since 1974, the University of Florida has been engaged, under EPA contract [6], in the aggregation of urban rainfall-runoff-quality data collected by others. These data are intended primarily for use in urban runoff model calibration and verification, but also may usefully serve to characterize urban runoff on a nationwide basis.

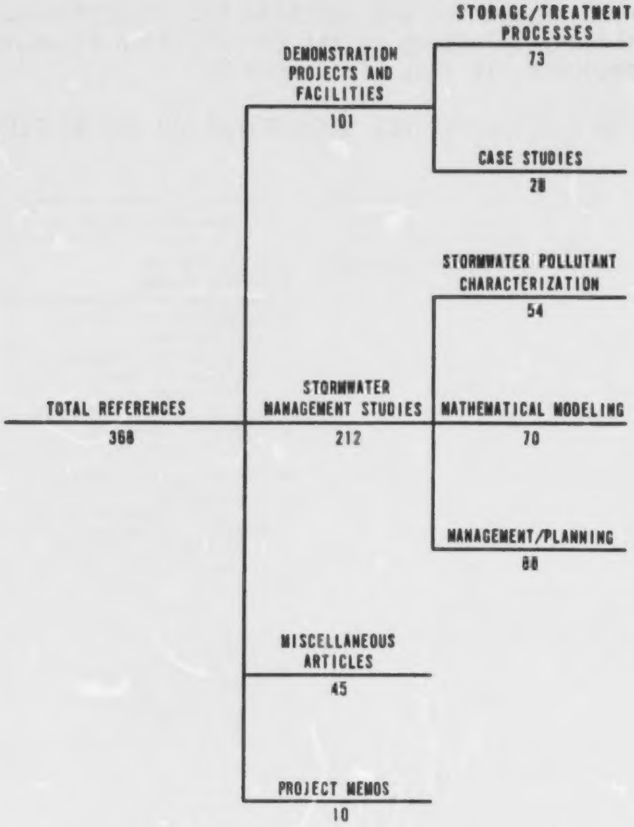

STORAGE/TREATMENT PROCESSES
73
DEMONSTRATION PROJECTS AND FACILITIES
101
CASE STUDIES
28
STORMWATER POLLUTANT CHARACTERIZATION
54
TOTAL REFERENCES
368
STORMWATER MANAGEMENT STUDIES
212
MATHEMATICAL MODELING
70
MANAGEMENT/PLANNING
88
MISCELLANEOUS ARTICLES
45
PROJECT MEMOS
10

Figure 1. Source reference distribution.

Locations for which data have currently been assembled and placed on magnetic tape are listed in the Appendix. Rainfall, runoff, and quality data are available for 7 locations while 12 additional locations have only rainfall-runoff data at present. Data are provided on a storm event basis: no long-term (continuous) records are presently included. Receiving water data are also not included.

EPA encourages active use and expansion of this Data Bank. A magnetic tape containing the data will be mailed at cost to those who request it through Wayne C. Huber and James P. Heaney, Department of Environmental Engineering Sciences, University of Florida, Gainesville, Florida 32611.

In addition, it is known that there are many data sources already in existence plus potential feedback from many of the nearly 200 EPA Section 208 Areawide Waste Management Studies that may be suitable for inclusion in the Data Bank. As sources are developed periodic addenda in the form of summary reports and tape updates will be issued.

SECTION 2

CONCLUSIONS

In the 3 years since the completion and publication of the initial comprehensive assessment of urban stormwater management and technology [1], much has been published on data and methodology; many planning studies have been initiated; several demonstration projects have been completed or significantly advanced; and, most importantly, a number of major projects have reached the threshold of final design and implementation.

In terms of potential investment, a sampling of the latter projects is both staggering and reassuring.

- Chicago, Illinois - \$1.8 billion program to control combined sewer overflows, partially under construction with \$662 million worth of work to go under contract this year [2].
- San Francisco, California - \$1.5 billion program to control combined sewer overflows and upgrade existing treatment with over \$170 million in construction, advertised or awarded and projected total system operation by 1985 [3].
- Boston, Massachusetts - \$0.8 billion program of regionalized treatment upgrading and combined sewer overflow control and treatment to be fully implemented by 2000 [4].
- Rochester, New York - \$0.4 billion program to control combined sewer overflows, expected to go under design this year [2].

The figures are staggering because these four metropolitan areas comprise only 15% of the nation's population served by combined sewers. Reassurance comes from the fact that these cities, on the cutting edge in stormwater management, have the confidence in today's new technology to move beyond the frustrating years of study into beneficial and broad scale implementation. The impetus of design and construction works on this vast scale will greatly accelerate our base of knowledge and implementation capabilities.

Conclusions with respect to the present level of urban stormwater management technology follow in the sequences as addressed in the body of the report.

APPROACH METHODOLOGY

The basic approach concept may be viewed as a four step process: (1) quantifying the need, (2) selective field monitoring, (3) cost-effectiveness assessment, and (4) impact simulations.

- Tools for analysis range from relatively simple desktop procedures to highly complex digital computer simulations. Of the available guides, the EPA-MERL guide [5] promises the greatest utility for the user.

- Models are available in four application categories as shown in Table 2. There are considerable variations in model complexity and utility within each level.

TABLE 2. LEVELS OF STORMWATER MANAGEMENT MODELS

Analysis level	Model type	Model complexity	Purpose of model	Model characteristics
I	Desktop	Low to medium	Problem assessment, preliminary planning, alternative screening	No computers. Equations, nomographs based on statistical analyses of many years of records.
II	Continuous simulation	Low to medium	Problem assessment, planning, preliminary sizing of facilities (particularly storage), alternative screening. Assess long-term impacts of designs.	Program of few hundred to few thousand statements. Uses many years of rainfall records with daily time steps, or worst 2 years with hourly time steps. May include flow routing and continuous receiving water analysis.
III	Single event simulation	Medium to high	Analysis for design, detailed planning	Program to over 10 000 statements. Higher modeling precision, from rainfall through sewers, possibly to receiving waters. Short-time steps and simulation times. Fewer alternatives to be evaluated.
IV	Operational	Medium	Real-time coverage of sewerage systems	Uses telemetered rainfall data and feedback from sewer system sensors to continually make short-term predictions of system responses, and so produce control decisions during storms.

Desktop procedures may be adequate where a gross assessment is required of the relative loads of various sources and their impacts on water quality. Continuous, simplified models add the benefits of total system perspective for problem identification and assessment for advanced planning, while detailed models enable final design evaluations and postconstruction assessments.

- In the final analysis, however, there is no substitute for experienced professional engineering evaluation. The "least common denomination solution" does not exist whether it be aimed towards design storm selection, impact analysis, cost assessment, or public acceptability evaluation.

STORMWATER CHARACTERISTICS

Characteristics of particular interest to the designer-manager are: (1) sources of pollutants, (2) discharge "end of the pipe" loadings, (3) process residuals, and (4) receiving water impacts. A logical fifth category, beneficial reuse, is an emerging research need.

- Major stormwater pollutant sources and the related data base activities are shown in Figure 2.

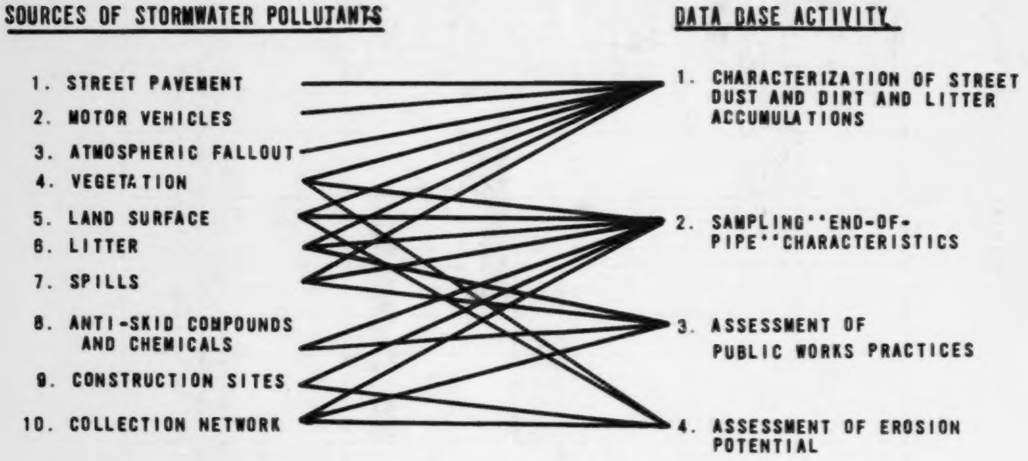

SOURCES OF STORMWATER POLLUTANTS
DATA BASE ACTIVITY
1. STREET PAVEMENT
2. MOTOR VEHICLES
3. ATMOSPHERIC FALLOUT
4. VEGETATION
5. LAND SURFACE
6. LITTER
7. SPILLS
8. ANTI-SKID COMPOUNDS AND CHEMICALS
9. CONSTRUCTION SITES
10. COLLECTION NETWORK
1. CHARACTERIZATION OF STREET DUST AND DIRT AND LITTER ACCUMULATIONS
2. SAMPLING "END-OF-PIPE" CHARACTERISTICS
3. ASSESSMENT OF PUBLIC WORKS PRACTICES
4. ASSESSMENT OF EROSION POTENTIAL

Figure 2. Relationships between sources of stormwater pollutants and data base activities.

- Stormwater discharge characteristics are becoming better defined, and, predictably, as the data base grows the spread between "average" values is significantly reduced. The reductions are apparent in Figure 3, which compares the present data base (from more than 2500 and 2200 samplings respectively, for separate stormwater discharges and combined sewer overflows) to those presented in the 1973 assessment [1]. Typical values are shown in Table 3.

- Data normalization has been performed by (1) system type, (2) mass (flow weighted) loadings, (3) land use, (4) precipitation and runoff characteristics, and (5) time (both within a specific event and as a function of intervals between events). A major deficiency in historical data is the absence of quantity-quality synchronization.

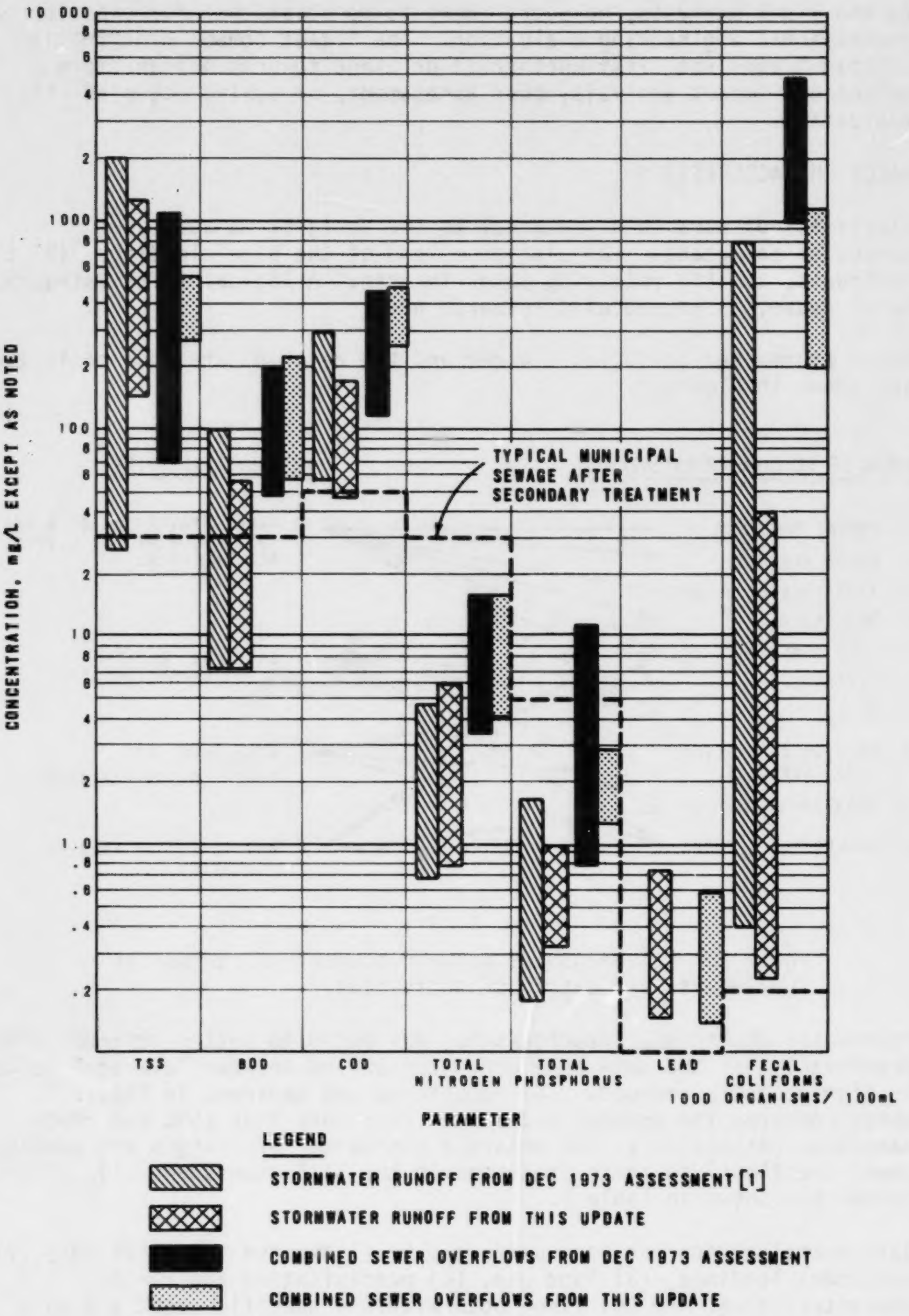


Figure 3. Representative stormwater discharge quality.

TABLE 3. COMPARISON OF TYPICAL VALUES FOR STORMWATER DISCHARGES[a]

	TSS	VSS	BOD	COD	Kjeldahl nitrogen	Total nitrogen	PO_4-P	OPO_4-P	Lead	Fecal coliforms
Background levels	5-100	...	0.5-3	20	.....	0.05-0.5[b]	0.01-0.2[c]	....	<0.1	....
Stormwater runoff	415	90	20	115	1.4	3-10	0.6	0.4	0.35	14 500
Combined sewer overflow	370	140	115	375	3.8	9-10	1.9	1.0	0.37	670 000
Sanitary sewage [6]	200	150	375	500	40	40	10	7	....	....

a. All values mg/L except fecal coliforms which are organisms/100 mL.

b. NO_3 as N.

c. Total phosphorus as P.

Results of data normalization by system type, parameter, and time into event are shown in Figure 4.

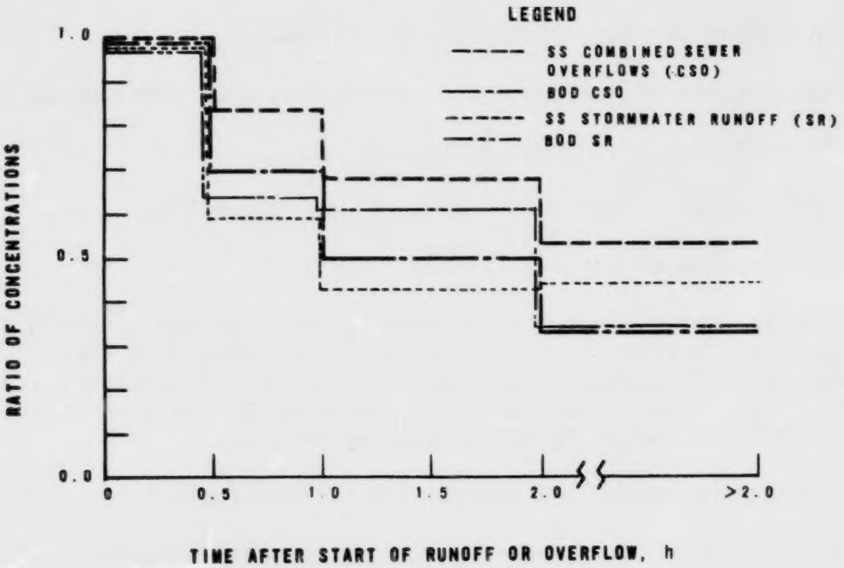


Figure 4. Time weighted normalization.

- Residual sludges from stormwater treatment processes are likely to be high in inorganics (volatile solids content about half that in raw primary sludge), treatable by conventional processes, but so great in volume as to provide major treatment and disposal problems. Further characterization and design experience are significant research needs.

- Receiving water impact evaluations to date must rely on model simulations due to the intermittency and variability of events and the masking effects of other discharges.

BEST MANAGEMENT PRACTICES

Nonstructural and low structurally intensive alternatives, termed best management practices (BMPs), offer considerable promise as the first line of action to control urban runoff pollution. By treating the problem at its source, or through appropriate legislation curtailing its opportunity to develop, multiple benefits can be derived. These include lower cost, earlier results, erosion/flood control benefits, and an improved and cleaner neighborhood environment.

- Problem prevention goals center about containment of all or part of the runoff and pollutants near the source. Planning elements include:

    1. Utilization of greenways and detention ponds
    2. Utilization of pervious areas for recharge
    3. Avoidance of steep slopes for development
    4. Maintenance of maximum land area in a natural undisturbed state
    5. Prohibiting development on flood plains
    6. Utilization of porous pavements where applicable
    7. Utilization of natural drainage features

- Construction controls such as minimizing the area and duration of exposure, protecting the soil with mulch and vegetative cover, increasing infiltration rates, and construction of temporary storage basins or protective dikes to limit storm runoff can significantly reduce receiving water impacts caused by erosion.

- Corrective maintenance and operation practices include:

    1. Control of litter, debris, and agricultural chemicals
    2. Regular street repair and sweeping
    3. Improved roadway deicing and materials storage practices

4. Proper use and maintenance of both catchbasins and drainage collection systems

5. Onsite retention or detention of stormwater runoff

- Program success is dependent on legislation or ordinances, to force or encourage conformance with the intended BMP, and a concerted effort to monitor compliance and educate not only those who will bear the responsibility of regulation, but the public as well.

- The greatest difficulty faced by BMP is that the action-impact relationships are almost totally unquantified. It is clear that onsite storage, for example, can be closely related to reduced downstream conduit requirements; but the net water quality benefits are far less defined. Similarly, cleaner streets and neighborhoods and enforced legislation will eradicate gross pollution sources, but to what limit should they be applied and who will bear the cost? The final answers of cost effectiveness have not been found short of trial implementation.

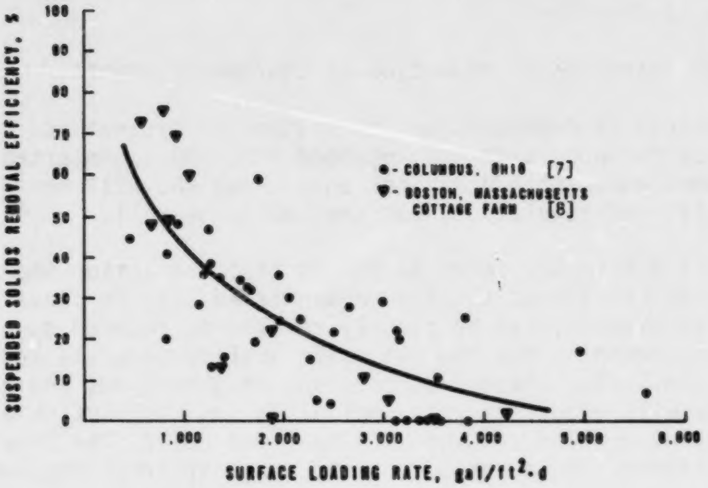
UNIT PROCESSES

The alternatives, or preferably supplements to BMP, involve combinations of storage and high-rate unit processes and/or conjunctive use with existing treatment facilities.

- Storage is considered a necessary control alternative because of the high volume and variability associated with storm and combined sewer overflows. Storage facilities are frequently used to attenuate peak flows, thereby reducing the size of facilities required for further treatment. Storage, however, with the resulting sedimentation that occurs due to increased detention times, can also be considered a treatment process, as shown in Figure 5.

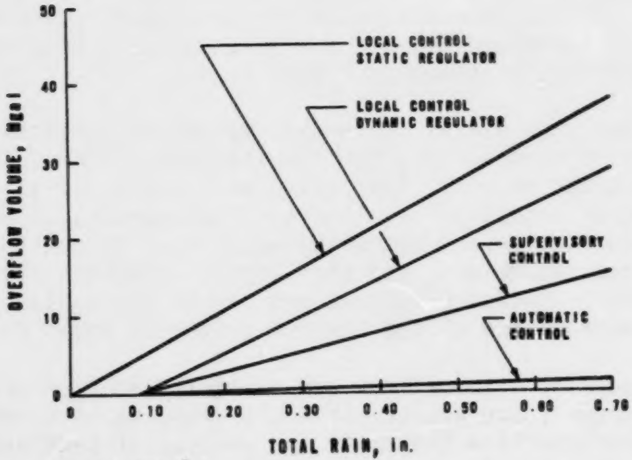
- Inline storage, the use of the unused volume in interceptors and trunk sewers to store runoff, is a particularly attractive option for controlling urban runoff. Typically, this alternative includes installation of effective regulators, level sensors, tide gates, rain gage networks, sewage and receiving water quality monitors, overflow detectors, and flowmeters, and then applies computerized collection system control. System effectiveness may be highly sensitive to the degree and maintenance of the control system as shown in Figure 6.

- Physical treatment alternatives are primarily applied for solids removal from wastestreams, and are of particular importance to storm and combined sewer overflow treatment for removal of settleable and suspended solids and floatable material. Physical treatment systems have demonstrated capability to handle high and variable influent concentrations and flowrates and operate independently of other treatment facilities, with the exception of treatment and disposal of the sludge/solids residuals. The principal disadvantage relates to those periods of time when equipment sits idle during periods of dry weather. When implemented on a dual-use basis as either pretreatment or



gal/ft²·d x 40.74 = L/m²·d

Figure 5. Typical suspended solids removal efficiencies for storage/sedimentation facilities without chemical addition.



Mgal x 3785=m³
in. x 2.54 = cm

Figure 6. Inline storage effectiveness regression lines for each mode of control (Seattle).

effluent polishing of conventional treatment plant flows, reduced capital investments may be realized. Representative process efficiencies and costs are shown in Table 4.

TABLE 4. COMPARISON OF PHYSICAL TREATMENT SYSTEMS

Physical unit process	Percent reduction						Average capital cost, $/Mgal·d[a]
	Suspended solids	BOD_5	COD	Settleable solids	Total phosphorus	Total Kjeldahl nitrogen	
Sedimentation							
Without chemicals	20-60	30	34	30-90	20	38	23 000
Chemically assisted	68	68	45	.....	..	..	23 000
Swirl concentrator/flow regulator	40-60	25-60	..	50-90	..	..	4 500
Screening							
Microstrainers	50-95	10-50	35	.....	20	30	19 500
Drum screen	30-55	10-40	25	60	10	17	19 300
Rotary screens	20-35	1-30	15	70-95	12	10	19 900
Disc screens	10-45	5-20	15	.....	..	..	......
Static screens	5-25	0-20	13	10-60	10	8	17 600
Dissolved air flotation[b]	45-85	30-80	55	93[c]	55	35	34 000
High rate filtration[d]	50-80	20-55	40	55-95	50	21	58 000
High gradient magnetic separation[e]	92-98	90-98	75	99	..	..	......

a. ENR Construction Cost Index 2000.

b. Process efficiencies include both prescreening and dissolved air flotation with chemical addition.

c. From pilot plant analysis [9].

d. Includes prescreening and chemical addition.

e. From bench scale pilot plant operation, 1 to 4 L/min (0.26 to 1.06 gal/min).

- The effects of chemical addition to enhance the physical removal properties have been demonstrated for most unit processes and generally show increased pollutant removals and a tolerance for higher hydraulic loading rates. Chemical addition to dissolved air flotation and high rate filtration processes have shown the greatest performance improvement, generally ranging to 20% and higher. Paced control of chemical additions continues to be a major problem, however.

- Of the physical processes, screening has received the greatest attention during the study period of this update. In general, performance in early prototype scale has been below expectations and unit costs significantly higher. Comparative screen performances are shown in Figure 7.

- Swirl concentrator/regulators have shown a steady and attractive solids removal performance over a wide range of hydraulic loading rates. Units have been demonstrated up to 3.6 m (12 ft) in diameter for design flows up to 300 L/s (6.8 Mgal/d). The swirl flow principle has also been successfully demonstrated as a grit separation device and as a primary (treatment) separator (effectiveness presently limited to relatively small diameter, 5.5 m [18 ft] units) [10, 11]. Investigations are proceeding on its potential use as a portable erosion/construction site treatment device [12].

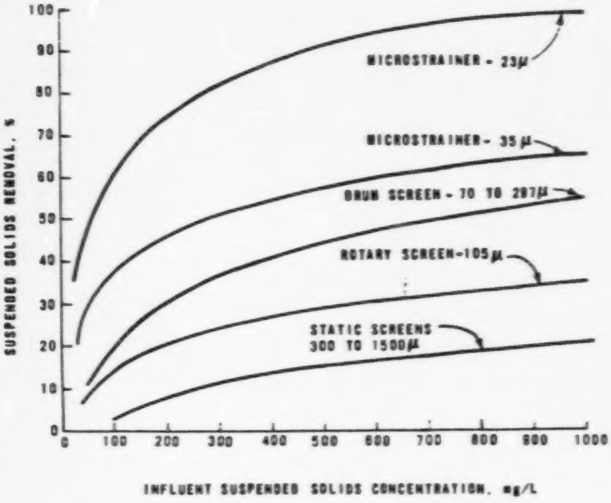


Figure 7. Comparative screen performance.

- Development and testing of new biological treatment processes and further demonstration of established stormwater biological systems at other locations have not been attempted beyond the originally reported demonstration facilities in reference [1].

- Land treatment of stormwater is limited by hydraulic application rates and the resulting land area requirements. Potentially promising processes include wetlands development, rapid infiltration, and overland flow. Conclusive design, operating, and performance data are unavailable. Marsh systems can handle the high solids loading associated with stormwater runoff and management techniques to increase pollutant removals are available from other field studies.

- Costs of disinfection systems used to treat combined sewer overflows and stormwater discharges can vary greatly depending on the complexity of the system. Stormwater disinfection must be flexible and capable of automatic operation to handle intermittent and varying flows and volumes.

High rate oxidizing agents, agent storage and handling, and high intensity mixing are major design considerations.

- In practice, operation and maintenance costs for stormwater facilities may show a marked economy of scale as shown in Figure 8. The illustration represents a cost to volume normalization of nine demonstration storage facilities.

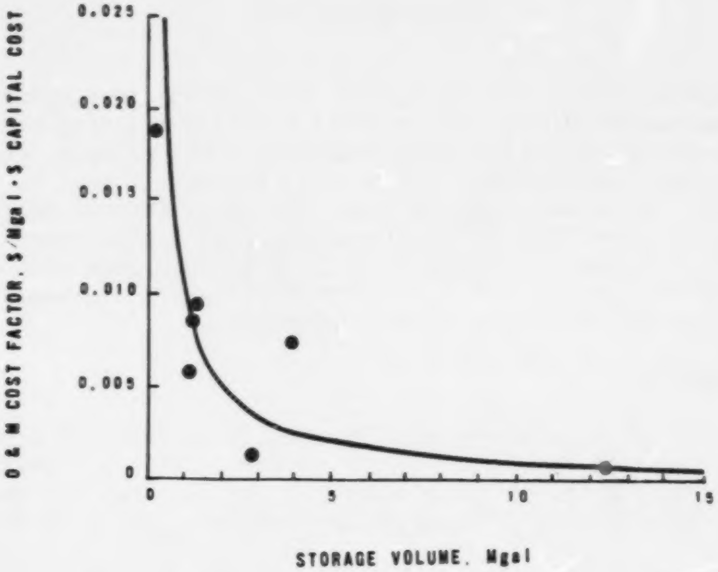


Figure 8. Operation and maintenance cost function for storage facilities.

SYSTEM APPLICATIONS

The size and complexity of urban runoff management programs are such that there is a need for an integrated approach to their solution. The solution is most often a combination of various best management practices and unit process applications.

- Regulatory constraints and public attitudes on pollution and environmental objectives are subject to change with time, thus mandating flexibility as a major program criterion.

- Demonstrated implementation progress to date is predominately in the areas of CSO control, excess flow treatment from heavily infiltrated sanitary systems, and BMP applications in new communities.

- Capital cost investments for structurally intensive alternatives commonly range from $3000 to $10 000/ha ($8000 to $24 000/acre) of sewered area. Degree of control varies from coarse screening and disinfection to complete secondary treatment with recreational reuse.

SECTION 3

RECOMMENDATIONS

The dollar investment in federally funded research and development (R&D) projects for stormwater management is dwarfed by the existing and potential construction costs generated off this data base. For example, Chicago's potential investment of $662 million in this, the first, year of its construction program is more than 10 times the total federal share in all storm and combined sewer R&D projects over the past twelve years since program inception. If the required technology base is to keep pace with or lead activities on such a scale, much greater emphasis must be given to the R&D effort with particular attention to the following.

IMPACTS AND BENEFITS

- Ties between receiving water quality and stormwater discharges must be clearly delineated in a wide variety of circumstances. This will require continuous monitoring of the selected discharge flows and the affected receiving water bodies to observe temporal and spatial impacts.

- Quantification of the impairment of beneficial uses and water quality objectives by such discharges should be a major criterion of these studies.

- As an alternative to direct discharge, beneficial reuse--the acceptance of urban runoff as a potential water resource--should be singled out as a prime R&D area. What are the potentials? What are the hazards? Why might runoff be preferred over other sources? How can reliability be built into designs to serve what specific uses? What is the cost outlook? For example, can highway drainage be ponded and reused for landscape irrigation?

- Renovated river systems, such as the San Antonio River in San Antonio, Texas, and the South Platte River in Denver, Colorado (Figure 9), should be fully researched and touted as clear evidence of documented benefits to be derived from improved stormwater management. These benefits include increased property values, alleviation of health and sanitation hazards, increased recreational facilities and linear parks, flood control, and enhanced community pride and quality-of-life [1].

- Finally, the potential beneficial reuses of the sludge by-products of stormwater treatment should be systematically evaluated. Is it suitable for direct landfill? If washed, will it prove to be a suitable foundation and grading material? Fine aggregate substitute? Cover material for refuse landfills?

Figure 9. South Platte River Renovation Project,
Denver, Colorado.

BEST MANAGEMENT PRACTICES

- Quantification of BMP action-impact relationships should be sought through multiple, broadscale, and regionally dispersed demonstration projects. Using runoff loadings, cost, and methodologies as primary criteria, comparisons of undeveloped versus developed and intensely applied BMP versus controlled no-action areas should be fully documented.
- Implementation, legislative, education, and enforcement experience gained from areawide studies on a national basis should be researched, consolidated, and published for local and subregional guidance and information.

UNIT PROCESSES

- The changes in stormwater characteristics as affected by storage should be analytically researched and published in a single subject document.

Specific interests concern odor generation, solids settlement and resuspension, and waste stabilization.

- The feasibility of pretreating and storing runoff in combined systems and subsequent reuse for system flushing should be investigated and demonstrated on a prototype scale.
- The dual-use application experience of new physical and, if applicable, biological stormwater treatment prototype systems should be researched in depth and published in a single subject document.
- The role of wetlands in the natural treatment of urban runoff and in the self-purification of streams should receive increased attention and information dissemination. Augmentation through applied land treatment technology should be investigated.

DATA MANAGEMENT AND DISSEMINATION OF INFORMATION

- Centralized storage and retrieval systems for stormwater quantity/quality and impact data, either regionally or nationally, are recommended as an adjunct to the essential free and rapid flow of priceless data between the researcher and the user.
- Information should be logged as quickly as possible and tagged with an identifier based on the degree of prescreening and verification of entries accomplished. All data not screened and verified within a specified time of posting, say 6 months, should be dropped from the system.
- All funded prototype demonstration projects should have a mandatory, preformatted, reporting requirement to the system on a monthly or bimonthly basis.
- When of significant program interest, funding for continuous monitoring and reporting beyond the normal project duration should be provided.
- Access to the data bank system should be open to anyone at nominal charge. Semiannual listings and updates of logged material should be published.
- Because of the continuing proliferation of publications in the field and the tendency towards rapid data obsolescence, universal assessments such as presented in this volume and its predecessor should give way to restricted subject area documents, thus permitting more in-depth analysis.

SECTION 4

APPROACH METHODOLOGY

The multivariable and complex nature of stormwater management assessments makes systematic approaches essential. Benefits to be derived from well structured and documented procedures include the identification of local data strengths and weaknesses, transferability of findings, and progressive adaptation to new technology and data sources. The information presented in this section provides the framework for the user to structure solution-oriented approaches and demonstrates their applicability through illustrative problem solving. Subsequent report sections provide data, management practice, unit process, and experience updates to be utilized in turning the approaches into practice.

INTRODUCTION

Two items have primary significance in framing approaches: (1) the basic concept and (2) the level of analysis required.

The Basic Concept

The basic approach may be viewed as consisting of four major steps.

Step 1. An effective approach methodology must be built on a quantified need. Thus, a logical first cut approach will intermix (1) known drainage area characteristics and hydrology, (2) reasonable ranges of pollutant-washoff and source potential, (3) background and direct discharge (point source) loadings, and (4) prevailing water quality conditions versus objectives. The purpose is to predetermine how much of what problem associated with what event frequency could be attributed to urban runoff dynamics.

Step 2. Selective field monitoring, guided by such analyses, should be concentrated in critical stream reaches and representative catchments. This second level investigation is necessary to substantiate the local applicability of assumed "best fit" data and to refine estimates.

Step 3. With the problem quantified and substantially isolated, a cost-effectiveness assessment of abatement alternatives has an improved likelihood of success. In this assessment, unit processes and improved management practices, singly or in combinations, are applied to the problem, costs established, and performance (benefits) quantified.

Step 4. Finally, repeat simulations of the receiving waters, loaded under post plan conditions, may be performed to yield a measure of the improvements potentially attainable.

Level of Analysis

The program for urban stormwater management for water quality benefits is a new and developing art. However, pressures for rapid, balanced control and restoration of receiving water quality have forced the program to center stage alongside the relatively mature programs of municipal and industrial wastewater treatment with several decades of experience behind them.

Fortunately, this condition has spawned several tools and methodologies for identifying and attacking stormwater problems, ranging from simple desktop procedures and nomographs to extremely complex computer simulations with 1 to 3 minute real time iteration cycles and with provisions for backwater, surcharging, looping, etc. Unfortunately, however, the core data on which quantitative assessments are built today are strikingly similar, marking the more complex approaches with a stigma of potential overkill.

Just as federal/state basin programs progress from waste load allocation (Section 303(e) [1]) through problem identification and assessment (Section 208) to facilities planning, design, construction, and operation (Section 201), so should the user be guided in his selection of tools and level of analysis. Desktop procedures may be entirely adequate for waste load allocations on the majority of streams that have minimal historical quality records. Simplified models add the benefits of total system perspective and time variability for advanced planning for problem identification and assessment. Finally, detailed models provide the consistency and precision for final design evaluations and post construction assessments. Each level of analysis and the applicable tools will be addressed in the body of this section.

CHARACTERIZATION OF THE NATIONAL PROBLEM

Under contract to the EPA, a joint effort of the American Public Works Association (APWA) and the University of Florida has recently produced a Nationwide Assessment of Combined Sewer Overflows, Urban Stormwater Discharges, and Nonsewered Urban Runoff [2]. The methodology used, the assumptions made, and problems encountered are of interest and potentially applicable on both regional and subregional analyses. Selected conclusions from that study follow:

- Dry-weather flows represent 30 to 50% of the total runoff from urban areas [the balance being wet-weather flows].
- Wet-weather organic loads from combined sewered areas are approximately four times higher than those from separate sewered areas.
- Loading rates [pollutant releases to receiving waters] for untreated dry-weather flow are higher than for wet-weather flow. However, if 90% secondary treatment is assumed for dry-weather BOD

generation, wet-weather loads are seen to be a significant portion of the total loadings from urban areas.

- A generalized method for evaluating the optimal mix of storage and treatment for wet-weather pollution load reduction indicated that a primary type facility is preferable up to BOD removals of about 10%. A secondary facility is preferable for higher levels of control.

- The "first flush" assumption, i.e., the assumed pollutant washoff rate, has a significant impact on the assessment. Control costs are about one-third less if a first flush is assumed.

- Incremental costs for wet-weather control increase significantly with higher control objectives. This is due to the disproportionately larger control units needed to capture the less frequent, larger storms.

- Significant savings might be realized [30 to 70%] by integrating wet-weather quantity control [storage] and dry-weather quality control [joint use of "excess" treatment capacity].

- Approximately 39% of the combined sewer problem and 10% of the other wet-weather flows should be controlled before initiating tertiary treatment control on a national average basis using BOD removal as the effectiveness measure.

- Runoff [wet weather] pollution can govern the quality of receiving waters due to the shock effect and long-term buildup of solids [benthic demands, turbidity, and smothering of attached aquatic growths and organisms].

An important additional finding of the study was the identification of the gross inadequacies of the present data base and the high sensitivity of the conclusions to the assumptions [ground rules] required for simulation. The major techniques applied and assumptions made are discussed in the following subsections and in Section 5.

PLANNING GUIDES

Guides to planning the management of urban stormwater quality may be divided into the following five principal components for the convenience of their users:

- Determination of stormwater quantity and quality at the outfall or interceptor

- Evaluation and selection of control alternatives

- Analysis of receiving water impacts

- Assessment of pollution control costs and benefits
- Examples of methodology applications

Among the documents published since January 1974, which are designed to aid managers, three have been noted for their fuller treatment of these five components, although on different levels of complexity and to varying extents. These guides are summarized in Table 5.

TABLE 5. SUMMARY OF PRINCIPAL PLANNING GUIDES

Title	SWMM: Level I - Preliminary Screening Procedures	Water Quality Management Planning for Urban Runoff	Areawide Assessment Procedures Manual
Prepared by	University of Florida	URS Research Company	EPA Municipal Environmental Research Laboratory
Release date	October 1976	December 1974	July 1976
Reference No.	3	4	5
Complexity level(s)	I-low	II-medium	III-low to high
Coverages			
• Discharge quality and quantity	Yes	Yes	Yes
• Control alternatives	Yes	General discussions, only	--[a]
• Receiving water impacts	No	No	Yes[b]
• Control costs and benefits	Yes	No	Not clear
• Example applications	Partial	Yes	--[a]

a. Announced as forthcoming.

b. Released after the completion of this study.

The features of the three principal planning guides are reviewed in more detail in the next three subsections. These reviews include discussions of the purposes for which they are suitable, noted omissions, and their state of completeness and usability, including the data and effort required to obtain results. Subsequent subsections will compare these three guides and cite other potentially helpful guides.

SWMM: Level I - Preliminary Screening Procedures (University of Florida) [3]

This report documents a simplified procedure to permit preliminary screening of alternative urban stormwater quality management plans, such as may be required by the 208 planning effort. Computers are not required by this procedure; the authors indicate that three more procedures using computers at increasing levels of complexity are to be released soon.

The primary objectives of this Level I desktop procedure are to compare stormwater storage and treatment, domestic wastewater, and stormwater management, on the basis of cost. Receiving water impacts are not evaluated.

The prospective user of this procedure should be warned that the documentation is not designed as a manual or users' guide, but instead reports all aspects of the procedure, including many considerations that went into its development. As a result, a planner or manager wishing to implement this procedure may find it difficult in places to identify and follow its progression.

The input data and principal computations required by the procedure are as follows:

> Information on land use (five types) in the study area, and land area and population per sewer type (three types), is first used to determine the population served and population density, by sewer type. Percent imperviousness is computed from population density. The annual runoff, including pervious area runoff, is then computed from both the mean annual precipitation and the computed percent imperviousness by a runoff coefficient method allowing for depression storage. The annual dry-weather flow is calculated from an average per capita flow and the computed population density.
>
> A street sweeping frequency, with previously obtained data on land use, population density, and type of sewer system, is next used to compute annual average loads of five different wet-weather pollutants. Dry-weather BOD loads, by type of sewer system, are computed from an average per capita generation rate.
>
> Cost functions, including operation and maintenance at 20% of the annual amortization cost, are provided for four primary wet-weather control devices (40% BOD removal assumed), two secondary devices (85% BOD removal assumed), and four types of storage (no treatment in storage is assumed). A series of computations based on the economic principles of production theory yield optimized total annual costs for combinations of storage and secondary treatment providing 25, 50, and 75% pollutant control. In this area of the document, the procedure to be followed by a user is very difficult to separate from the methods of its derivation. A graphical representation of the methodology is given in Figure 10. For different combinations of treatment rate and storage capacity (expressed as the depth of runoff contained over the entire drainage area), the isoquant curves in Figure 10 represent equal degrees of treatment. Isocost lines represent storage-treatment combinations which may be implemented at the same total cost. The point of tangency between an isoquant curve and an isocost line represents the most economical combination for a given degree of treatment. The optimum combination for any degree of treatment can be found from the "expansion path" through all tangent points.
>
> A relatively simple tradeoff analysis procedure is provided to compute the amount of wet-weather pollution that should be controlled before

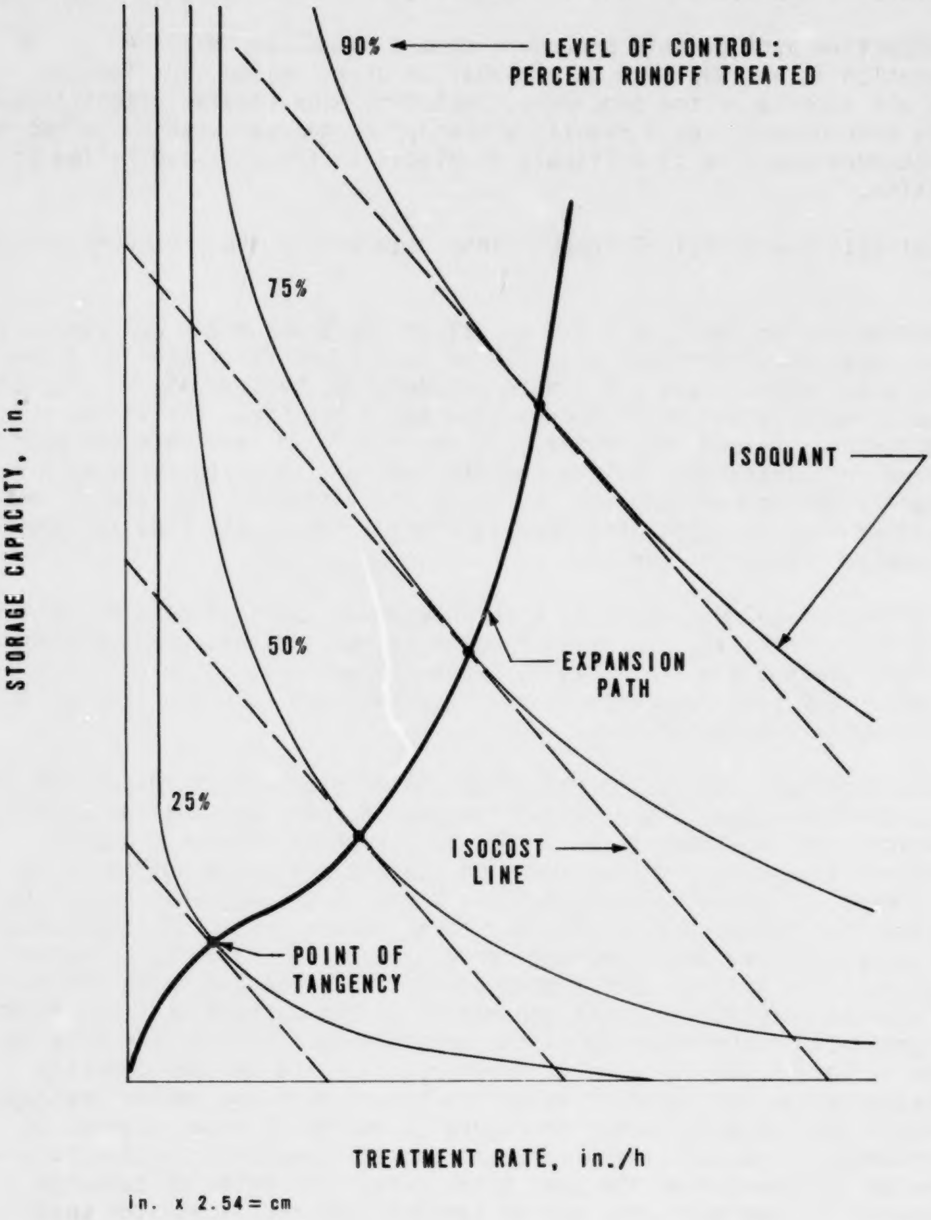


Figure 10. Determination of optimal combination of storage/treatment alternatives.

initiating tertiary treatment. Last, a procedure is given to roughly estimate the potential savings possible from multipurpose planning which integrates storage and treatment for wet-weather quality control with dry-weather sewage treatment plants and/or storage facilities for wet-weather quantity control.

A number of the methods used in this procedure were derived from the principles employed by, or from multiple runs made with, an earlier version of the STORM model [6]. In particular, this is the case for the runoff coefficient method, the street sweeping frequency relationship, the pollutant removal procedure, and the study of different storage/treatment configurations.

Example calculations for hypothetical cities are given [illegible] most steps of the procedure, except for the runoff quantity and quality analysis and for the potential savings due to multipurpose planning.

Water Quality Management Planning for Urban Runoff (URS Research Company) [4]

This earlier (December 1974) report is designed as a manual intended to enable planners to obtain a first approximation, or general, "first glance" assessment of the magnitude of the stormwater management problem. It is not intended to be used as a basis for abatement design; the broad objective of the method is to reduce discharge pollution. Dry-weather flows are not accounted for. The manual is organized into two sections, which (1) help quantify the local urban nonpoint water pollution problem without extensive data generation, and (2) make preliminary evaluations of cost-effective abatement and control practices. Computers are not required to be able to follow this manual; however, its desktop procedures are considerably more thorough and sophisticated than those presented in SWMM-Level I.

In the first section, three different levels of assessment (I, II, III) are suggested for pollution quantification, each requiring more input data and analysis and, therefore, each providing more sophistication and accuracy. Water quality and quantity may be obtained as functions of time using Levels II and III, which is especially important for identifying the most polluting parts of the runoff and for treatment, storage, and/or disposal.

In the second section, information is given about various abatement, treatment, and disposal methods, but no guidance is provided as to how this information should be used in making planning and management decisions. The authors state that the material in this section is not intended to serve as design criteria.

No procedures are provided in this manual for considering receiving water impacts, although the subject is discussed briefly in a separate, short, state-of-the-art review. Costs are not considered or mentioned other than for examples of some of the treatment methods. Two deficiencies noted in the EPA Review Notice are: (1) the design storm is selected without consulting local precipitation or hydrologic data; and (2) pervious area runoff is assumed to be zero, when it may in fact be significant.

Input data and principal computations required by the procedures are as follows:

First, information on the following study area characteristics, by subarea if these differ significantly, is collected: area size, amounts of street and "interconnected impervious" surfaces, and length and slope of the main drainage channel.

Considering an average street sweeping frequency, or the time elapsed since the last half inch of rain, contaminant surface accumulation rates and materials composition are next determined by one of three methods of increasing sophistication:

Level I -	using tables of national daily averages
Level II -	using tables of daily averages for broad categories and statistical confidence levels
Level III -	using site specific data, either from site surveys or from the literature

In order to compute runoff, a design storm is first selected; a "typical" 30-minute storm, likely to occur two or more times per year is recommended. Selection considerations and implications are discussed. A runoff coefficient method is then used to compute the resulting runoff rate.

The volume of runoff from impervious areas for the 30-minute storm is calculated from a simple formula. Unit hydrographs, based on Espey correlations, are then synthesized separately for interconnected pervious surfaces and for street surfaces only. These are then modified to reflect the specific precipitation rate.

Three alternative procedures of increasing sophistication are provided to assess the runoff quantity and quality at the outfall from the entire study area. These are:

Level I -	simply compute the average concentration per storm from the volume of impervious runoff and the amount of street contaminants removed.
Level II -	the time varying quality of the runoff (both pollution concentration and cumulative load) at the outfall is computed by a repetitive procedure for incremental time-steps across the hydrograph, similar to that employed within the Storm Water Management Model [7].
Level III -	the method of Level II is used to determine the quantity/quality hydrographs at the outlet from each subarea; the various flow lag times to the study area outfall are determined from a nomograph and a

table; and then the lagged hydrographs from the various subareas are combined by addition.

The quality composition of the resulting discharge is determined by proportionality to the total solids content.

The guidance provided by this manual in the areas of treatment, abatement, and disposal methods does not take the form of a procedure, so no specific input data are required. Instead, general information is provided on each of the following:

1. Source abatement (litter, public incentives, city management)
2. Technical abatement (improved street cleaning practices, air pollution controls)
3. Stormwater treatment
   - Storage and associated treatment [8]
   - Physical treatment (screening, high and ultrahigh rate filtration, swirl concentrator)
   - Biological treatment (high rate trickling filters, contact stabilization)
   - Physical-chemical treatment
4. Alternative methods of disposal (spray irrigation, infiltration ponds).

A series of ten example calculations, mostly for hypothetical study areas, is included. These examples cover all the steps of the runoff quality/quantity determination procedure.

Areawide Assessment Procedures Manual (EPA Municipal Environmental Research Laboratory) [5]

This document is specifically designed to guide the planning and management of stormwater quality, with particular emphasis on urban stormwater. It also covers municipal and industrial, agricultural, and rural water pollution control. From the description of its proposed completed coverage, and from the portions released (Mailing No. 1) prior to this writing, it will clearly be the most comprehensive and organized guide of those reviewed.

The procedures of this manual appear to be strongly based upon receiving water impacts. They are provided at several levels of planning sophistication. Simple procedures are recommended for initial analysis to develop the insight and problem understanding to guide the application of the more complex techniques, where required. It assumes that the simplest techniques can often produce the necessary information that is to be used, and it stresses practical decision making. Computer simulation is used by the more advanced procedures of this manual.

In the main, the manual does not provide a detailed, step-by-step procedure, but rather it discusses comprehensively the many factors involved and techniques available. While it will, as a result, require considerably more study and understanding for successful application, its use should provide the most reliable results.

The portions of the manual presently available deal with the preliminary assessment of the problem and with procedures for assessing urban pollution. The types of input data and principal activities required by these procedures are as follows:

> Fundamental information necessary to characterize the planning area (point sources, land use, hydrology, receiving waters) and the water quality are first described.
>
> Preliminary procedures for identifying and assessing problems are treated next. Methods of waste source identification and evaluation, and receiving water analysis, are thoroughly discussed, and a hypothetical illustrative example of a preliminary impact analysis is given.
>
> Alternative detailed procedures to assess the magnitudes of pollutant loads are discussed comprehensively. First, the level of sophistication required must be identified, and three levels of refinement are suggested for defining transient stormwater loads. These are: average yearly storm load, actual event distribution, and variation within events. Second, the continuous point source loads (municipal, industrial) must be characterized, and the need for monitoring the sources determined. Third, the intermittent point source loads must be characterized, and, because of their variability, these require unique estimating procedures. For this purpose it is essential to collect, if necessary, and analyze local data on rainfall (most important), land use, and conveyance systems. After selecting an estimation procedure that will provide a suitable level of accuracy and detail, a rainfall-runoff relation and the runoff quality must be characterized. A runoff coefficient is suggested for the former and local monitoring, if possible, is recommended for the latter. With these estimates in hand, the storm loads may be estimated and assessed. Two particular methods are presented. The first is a simplified statistical method, for which the algorithms are given; it does not treat routing. An example application to 25 years of records at Denver is described. The second assessment method employs continuous computer simulation, such as that provided by the models STORM and SWMM (continuous version). The simplified SWMM developed by Metcalf & Eddy [9] is discussed in particular, including an example application to Rochester, N.Y.

Portions of the manual treating procedures for assessing nonurban pollution and analyzing receiving water impacts were released after the completion of this study; other portions to be released soon will treat procedures for evaluating and selecting control alternatives, and will provide examples of the two pollution assessment procedures. When complete, this document will undoubtedly stand as a key reference on 208 (PL 92-500 areawide waste treatment management) planning, for which it is specifically designed, and it can be expected to significantly aid 201 (facilities) planning.

Other Guides

Methodology for the Study of Urban Storm Generated Pollution and Control--

Probably the next most important document in the field is this report by Envirex, Inc. [10]. Its objective was to develop standard procedures needed to ensure that all stormwater and combined sewage discharges and treatment processes could be evaluated by the same means. Therefore, rather than being another planning guide, this report complements the existing planning guides in two important areas: field and laboratory procedures for assessing both runoff quality/quantity and treatment processes.

In the first of these areas (Section VIII of [10]), standard procedures are recommended for evaluating stormwater treatment processes, both for prototype installations and demonstration units. The types of treatment process application data needed to be able to compare applications at different sites are recommended, both in general and for specific systems. Uniform methods of reporting these data are also proposed, together with standard methods for describing facilities and reporting their costs. Different types of efficiencies which may be determined to evaluate and compare treatment systems are described. These depend greatly upon the bypass routes and durations, and include the volumetric efficiency (treated volume, V_5, divided by total volume, V_3, during the bypass period--refer to Figure 11), the overall efficiency (amount of pollutants removed, M_3-M_8, divided by amount of pollutants delivered, M_3), and the treatment efficiency (amount of pollutants removed, M_5-M_6 divided by amount of pollutants delivered to treatment, M_5). Such efficiencies should be obtained and compared for both individual storms and for long-term periods of a year or more.

Economic considerations for decision making are also reviewed in Section VIII, based upon the variation of receiving water quality with discharge abatement investment. Tradeoffs between desired receiving water quality and allowable economic investments will be necessary and these should consider all alternatives including integrated dry-weather and wet-weather systems, tertiary dry-weather treatment, land management, nonstructural techniques, and flood and erosion control integrated with pollution control. To make such decisions, performance curves for various control investments, of the type presented in Figure 12, are proposed.

In the second area of this document, many standard procedures are recommended for executing a stormwater and combined sewage management study. The advice contained is very practical, being based on field and instrument manufacturing experience and demonstration programs. It covers the following topics:

- Sampling procedures and considerations (sample location, frequency, type, size, number, containers, storage; use of automatic samplers)
- Monitoring instrumentation (flow gaging; methods of constituent monitoring; rain gages)
- Quality constituents, laboratory procedures
- Describing storm discharges (parameters; categories of monitoring programs, monitoring needs of each).

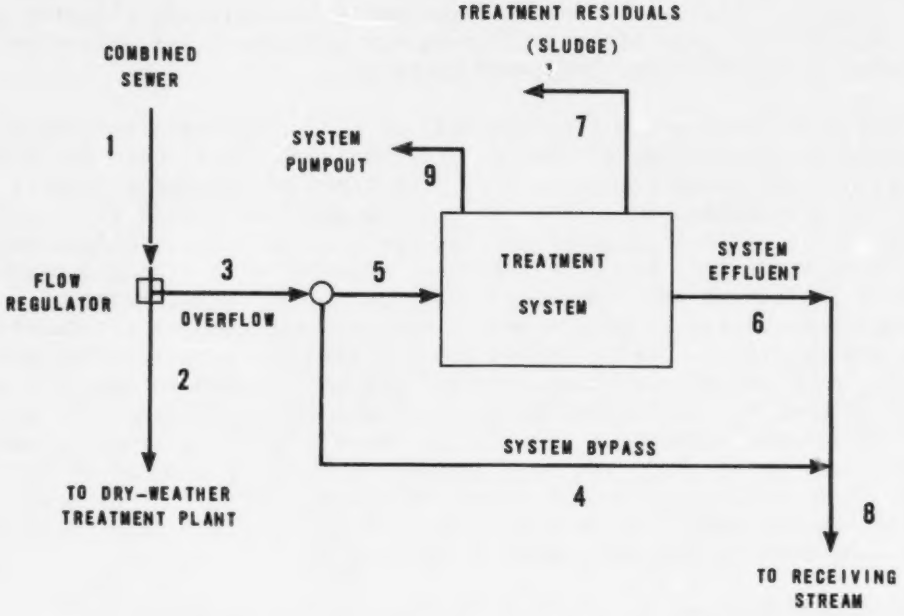
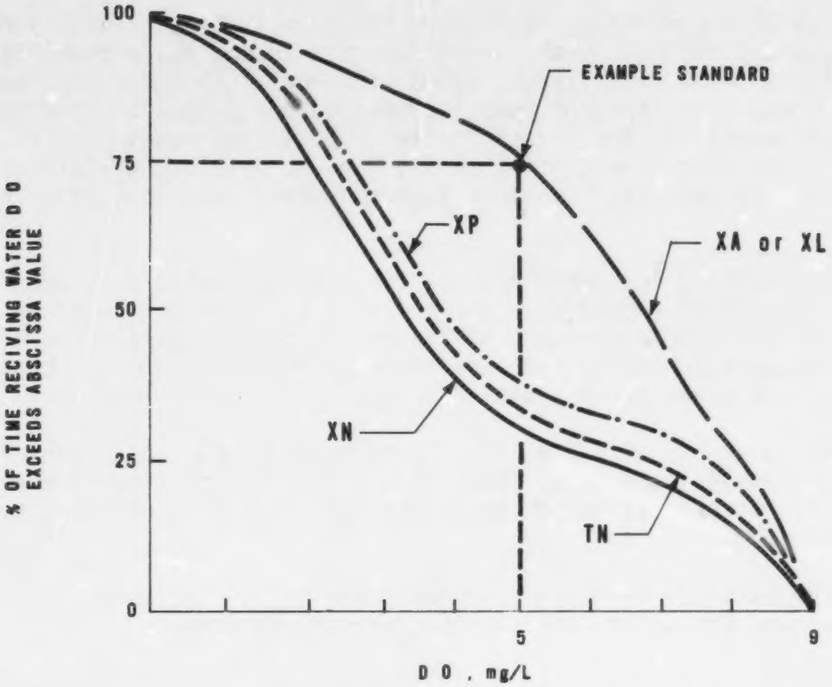

TREATMENT RESIDUALS (SLUDGE)
COMBINED SEWER
1
7
SYSTEM PUMPOUT
9
FLOW REGULATOR
3
OVERFLOW
5
TREATMENT SYSTEM
SYSTEM EFFLUENT
6
2
SYSTEM BYPASS
4
TO DRY-WEATHER TREATMENT PLANT
8
TO RECEIVING STREAM

Figure 11. Generalized combined sewer overflow treatment system [10].

Cost Estimating Manual - Combined Sewer Overflow Storage and Treatment--

This report by Culp, Wesner, Culp Consulting Engineers [11] contains the best present collection of cost data of its type. National average costs are provided for fourteen different process functions, ranging in capacity from 220 to 8800 L/s (5 to 200 Mgal/d), and for storage facilities ranging in size from 4 to 910 ML (1 to 240 Mgal).



CURVE	CONTROL ALTERNATIVES (BOD REMOVAL, %)		ADDITIONAL COST, MILLION $
	DRY WEATHER	WET WEATHER	
XN	EXISTING (85)	NONE (0)	0
TN	TERTIARY (95)	NONE (0)	8
XP	EXISTING (85)	PRIMARY (25)	1
XA	EXISTING (85)	ADVANCED (75)	6
XL	EXISTING (85)	PRIMARY AND LAND MANAGEMENT (75)	3

Figure 12. Hypothetical example of the economic solution methodology approach [10].

Interim Report on Loading Functions for Assessment of Water Pollution From Nonpoint Sources--

This November 1975 report by the Midwest Research Institute [12] is designed as a handbook for estimating the generation (or accumulation rate) of pollutant loads for nine different types of nonpoint sources, plus background pollution loads. The types of nonpoint sources included are: sediment from soil erosion, nutrients and organic matter, pesticides, salinity in irrigation return flow, acid mine drainage, heavy metals and radioactivity, urban and related sources, livestock in confinement, and leachates from landfill.

The loading functions for urban and related sources employ a subset of the pollution load values reported in the previously reviewed URS manual [4]. The functions are simple products of terms. The street surface loadings represent accumulation rates before removal by street cleaning. Example calculations are given for Atlanta, Georgia.

Similar information is also given for contributions from motor vehicles only, and from pavement deicing salt. The user is referred to other literature for methods to assess the effects of these surface loadings upon urban runoff quality.

Report to the National Commission on Water Quality on Assessment of Technologies and Costs for Publicly Owned Treatment Works--

A simplified method to determine the most economical mix of treatment rate and storage is provided in this report by Metcalf & Eddy, Inc. [13]. This method is based on an estimate of the volume required to store the runoff from a 2-year, 1-hour storm, and the treatment capacity required to process the captured volume in 24 hours. The required storage volume in cubic metres (Mgal) can be expressed by

$$99.9\ KiAt_1 - 3601\ Qt_2 \qquad \text{(SI units)} \qquad (4\text{-}1a)$$

or

$$0.02715\ KiAt_1 - \frac{Q}{24}\ t_2 \qquad \text{(U.S. customary units)} \qquad (4\text{-}1b)$$

where K = runoff coefficient
i = rainfall intensity (which varies by region), cm/h (in./h)
A = service area, ha (acres)
t_1 = rainfall duration, hours
Q = treatment process capacity required to treat captured volume in 24 hours, m^3/s (Mgal/d)
t_2 = runoff duration, hours

The first term in this equation is the total runoff to be stored; the second term is the amount of runoff treated during the storm.

The total cost of treatment and storage can be expressed by

$$99.9\ KiAt_1C_2 + Q[C_1 - 3601\ C_2t_2] \qquad \text{(SI units)} \qquad (4\text{-}2a)$$

$$\text{or} \qquad 0.02715\ KiAt_1C_2 + Q\left[C_1 - \frac{C_2t_2}{24}\right] \qquad \text{(U.S. customary units)} \qquad (4\text{-}2b)$$

where C_1 = unit treatment cost

C_2 = unit storage cost for a buried basin, which averages about \$264/m^3 (\$1 million/Mgal)

As a result, the optimum strategy is to minimize the treatment capacity Q for unit treatment costs greater than 3601 C_2t_2 and to maximize the treatment capacity for C_1 less than 3601 C_2t_2 (SI units).

Comparison of Guides

Technically, the three principal guides reviewed are only fully comparable for their computations of discharge quality and quantity. This comparison is summarized in Table 6.

TABLE 6. URBAN RUNOFF PROCEDURES OF THREE GUIDES

Guide	Quantity procedure	Quality procedure
University of Florida [3]	Runoff coefficient, annual average	From population density
URS Research Company [4]	Design storm and unit hydrographs	From tables or local monitoring
EPA-MERL [5]	Runoff coefficient, three levels	From local monitoring

Computers are needed only for the more complex procedures of the EPA-MERL guide [5]. Simple procedures are provided by both the University of Florida guide [3] and the EPA-MERL guide; the URS manual is generally intermediate in complexity [4].

Procedures for nonurban sources and areas are included in the Midwest Research Institute report [12], and in Chapter 4 of the comprehensive EPA-MERL guide.

As regards user convenience, the University of Florida guide is very difficult to follow in a number of areas; the URS manual contains some transposed sequences which may be confusing on first contact. Only the first half of the URS manual approaches being a step-by-step procedural guide, the other documents are primarily discussions of the various steps which should comprise management planning for urban stormwater quality.

MATHEMATICAL MODELS

An increased awareness of the significance of stormwater pollution has recently generated more interest in integrated dry-weather and wet-weather pollution control systems and flood and erosion control. The addition of these planning methods has greatly added to the complexity of the analyses required, and to the desirability of repetitive simulation procedures, particularly computer models, to evaluate the many management options.

Many new models for the assessment, planning, design, and control of storm and combined sewage systems have been released since 1973, and a number of reviews and comparisons of these models have already been published [5, 14-18]. The variety of complexities, capabilities, and intended purposes of these models has suggested various categorization schemes.

The available models are summarized and model application is discussed in the following paragraphs. The intent of this subsection is to present a methodology for model use, rather than provide a detailed description of the models themselves and their capabilities. Examples of model applications are included in the following subsection.

Available Models

Urban stormwater management models can be classed into the following four application categories:

1. Problem assessment
2. Planning
3. Event analysis
4. Operational

The first three categories tend to blend into one another, on an ascending scale of complexity and detail. The purposes and characteristics of these four fundamental types of model are summarized in Table 7; there are considerable variations in model complexity within each of these types or levels. While certain Level III models could provide continuous simulation, for example, they should not be considered members of Level II if their high complexities require overly large amounts of data and execution time.

Operational models, in the fourth category, are more distinct in nature from the other planning/analysis models. Of moderate to advanced complexity, they are used to produce control decisions during storms, most commonly for regulator operation (overflow control), but also for in-system storage and diversions. Rainfall data are entered from telemetered stations, and the model continually predicts in real time system responses a short time into the future.

TABLE 7. LEVELS OF STORMWATER MANAGEMENT

Analysis level	Model type	Model complexity	Purpose of model	Model characteristics
I	Desktop	Low to medium	Problem assessment, preliminary planning, alternative screening	No computers. Equations, nomographs based on statistical analyses of many years of records.
II	Continuous simulation	Low to medium	Problem assessment, planning, preliminary sizing of facilities (particularly storage), alternative screening. Assess long-term impacts of designs.	Program of few hundred to few thousand statements. Uses many years of rainfall records with daily time steps, or worst 2 years with hourly time steps. May include flow routing and continuous receiving water analysis.
III	Single event simulation	Medium to high	Analysis for design, detailed planning	Program to over 10 000 statements. Higher modeling precision, from rainfall through sewers, possibly to receiving waters. Short-time steps and simulation times. Fewer alternatives to be evaluated.
IV	Operational	Medium	Real-time coverage of sewerage systems	Uses telemetered rainfall data and feedback from sewer system sensors to continually make short-term predictions of system responses, and so produce control decisions during storms.

The basic capabilities of 29 major models, largely based on recent reviews [5, 14, 15], are summarized in Table 8. For the convenience of the prospective users, the models are organized into the previously discussed categories, in order of increasing sophistication.

A few of the computer models in Table 8 are listed under more than one level, because they have multiple capabilities. A number of the models are seen to have no capabilities to simulate water quality. Although the objective of stormwater management is to control pollution, certain supporting analyses require quantity simulation only. An example would be for making an initial study of storage/treatment tradeoffs, when sizing wet-weather treatment facilities for a dual wet-/dry-weather treatment system.

Certain of these computer models stand out for various reasons, including their availability and success in applications. Fifteen were found to provide few additional special features, and therefore were not recommended for routine application except in special circumstances [15]. The selected models are identified in Table 8 with asterisks. Brief descriptions, largely drawn from the above mentioned reviews [5, 14, 15], are provided for each of the selected computer models, generally in order of increasing complexity in

TABLE 8. CHARACTERISTICS OF PLANNING, DESIGN, AND OPERATIONAL MODELS, BY LEVEL, IN ORDER OF INCREASING COMPLEXITY

Model		Catchment hydrology						Sewer hydraulics						Wastewater quality							Miscellaneous				
Origin	Acronym	Multiple catchment inflows	Dry-weather flow	Input of several hyetographs	Snowmelt	Runoff from impervious areas	Runoff from pervious areas	Flow routing in sewers	Upstream and downstream flow control	Surcharging and pressure flow	Diversions	Pumping stations	Storage	Dry-weather quality	Stormwater quality	Quality routing	Sedimentation and scour	Quality reactions	Wastewater treatment	Receiving water quality simulation	Receiving water flow simulation	Can choose time interval	Design computations	Applied to real problems	Computer program available
LEVEL I (Desktop)																									
* University of Florida	SWMM Level I		▨			▨	▨						▨	▨	▨				▨				▨		
* URS Research Company	--					▨		▨							▨	▨									
* EPA-MERL	--		▨			▨	▨						▨	▨	▨			▨	▨	▨	▨		a	a	
LEVEL II (Continuous)																									
University of Massachusetts	--	▨	▨			▨		▨	▨																▨
Chicago Sanitary District	FSP	▨	▨	▨	▨	▨	▨	▨					▨								▨	▨		▨	▨
University of Illinois	ISS	▨	▨		▨	▨	▨	▨	▨		▨	▨	▨									▨	▨		▨
* Metcalf & Eddy	Simplified SWMM	▨			▨	▨	▨						▨		▨				▨	▨	▨	▨		▨	▨
Norwegian Water Res.	NIVA	▨	▨					▨	▨	▨	▨	▨	▨	▨	▨	▨			▨			▨	▨		
* Corps of Engineers	STORM				▨	▨	▨				▨		▨		▨			▨						▨	▨
City of Chicago	CHM-RPM	▨	▨	▨		▨	▨	▨		▨	▨			▨	▨				▨		▨	▨	▨	▨	▨
* Hydrocomp	HSP	▨	▨	▨	▨	▨	▨	▨	▨		▨		▨	▨	▨	▨		▨		▨	▨	▨		▨	
* Dorsch Consult	QQS	▨	▨	▨		▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨			▨	▨	▨	▨		▨	
LEVEL III (Single event)																									
Queen's University	OUURM	▨				▨	▨	▨																	
University of Nebraska	HYDRA	▨		▨		▨	▨	▨														▨			
Wilsey and Ham	WH-1	▨				▨	▨	▨														▨	▨	▨	
Colorado State University	--	▨	▨			▨		▨	▨																▨
University of Massachusetts	--	▨	▨			▨		▨	▨																▨
University of Cincinnati	UCUR	▨				▨	▨	▨													▨	▨			▨
British Road Research Lab	RRL	▨	▨	▨		▨		▨													▨	▨		▨	▨
Illinois State	ILLUDAS	▨	▨	▨		▨	▨	▨					▨									▨	▨		▨

TABLE 8. (Concluded)

Model		Catchment hydrology						Sewer hydraulics						Wastewater quality							Miscellaneous				
Origin	Acronym	Multiple catchment inflows	Dry-weather flow	Input of several hyetographs	Snowmelt	Runoff from impervious areas	Runoff from pervious areas	Flow routing in sewers	Upstream and downstream flow control	Surcharging and pressure flow	Diversions	Pumping stations	Storage	Dry-weather quality	Stormwater quality	Quality routing	Sedimentation and scour	Quality reactions	Wastewater treatment	Receiving water quality simulation	Receiving water flow simulation	Can choose time interval	Design computations	Applied to real problems	Computer program available
LEVEL III (Cont.) (Single event)																									
Chicago Sanitary District	FSP	▨	▨	▨	▨	▨	▨	▨					▨								▨	▨		▨	▨
CH2M-Hill	SAM	▨	▨	▨		▨	▨	▨		▨	▨	▨										▨			
* MIT - Resource Analysis	MITCAT	▨	▨	▨	▨	▨	▨	▨		▨	▨		▨								▨	▨		▨	
University of Illinois	ISS	▨	▨		▨	▨	▨	▨	▨		▨	▨	▨									▨	▨		▨
Norwegian Water Res.	NIVA	▨	▨					▨	▨		▨	▨	▨	▨	▨	▨			▨			▨	▨		
City of Chicago	RPM	▨	▨	▨		▨	▨	▨		▨	▨			▨	▨				▨		▨	▨	▨	▨	▨
* SOGREAH	CAREDAS	▨	▨			▨	▨	▨	▨	▨	▨	▨	▨			▨				▨	▨	▨		▨	
* Hydrocomp	HSP	▨	▨	▨	▨	▨	▨	▨	▨		▨		▨	▨	▨			▨		▨	▨	▨		▨	
* Battelle Northwest	BNW	▨	▨	▨		▨	▨	▨		▨	▨		▨	▨	▨	▨			▨			▨	▨	▨	▨
Watermation	CSM	▨	▨	▨		▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨			▨			▨	▨	▨	
* EPA	SWMM	▨	▨	▨		▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨
* Dorsch Consult	HMV-QQS	▨	▨	▨		▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨			▨	▨	▨	▨		▨	
* Water Resources Engineers	STORMSEWER	▨	▨	▨		▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨		▨		▨	▨	▨		▨	
[a]EVEL IV (Operational)																									
Minneapolis-St. Paul	UROM-9	▨	▨	▨		▨	▨	▨			▨											▨		▨	▨
* Seattle Metro	CATAD	▨	▨	▨		▨	▨	▨	▨	▨	▨	▨	▨									▨		▨	▨
* Battelle Northwest	BNW	▨	▨	▨		▨	▨	▨		▨	▨		▨	▨	▨	▨			▨			▨	▨	▨	▨
Watermation	CSM	▨	▨	▨		▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨			▨			▨	▨	▨	
* Hydrocomp	HSP	▨	▨	▨	▨	▨	▨		▨		▨		▨	▨	▨	▨		▨		▨	▨	▨		▨	

* Described in more detail in text.

▨ Characteristic included in model.

a Announced as forthcoming

the following discussion. Desktop models and procedures (Analysis Level I) not requiring computers are not included since they were described previously under Planning Guides.

Simplified Stormwater Management Model--

The Simplified Stormwater Management Model, developed by Metcalf & Eddy, Inc. [9], is an inexpensive, flexible tool for planning and preliminary sizing of stormwater facilities. It consists of a combination of hand computation procedures and seven small computer programs totaling only a few hundred statements. Together, these provide continuous, time-varying simulation, and statistical characterizations. Without simulating transport, the model assesses the storage-treatment balance, evaluates the quantity and quality of potential overflows, and may be used conjunctively with most receiving water models. It does not treat water quality other than at overflows. Overflow quality and all costs are computed by hand, and so any number of quality constituents may vary with time in any realistic way. It requires the following data: the characteristics of the overflow structure(s), general characterizations of the drainage (sub)areas and the interceptors, treatment rate and storage volume, monitored data on overflow quantities and quality, and long-term hourly rainfall records, preferably for at least 20 years.

Corps of Engineers STORM Model--

The Storage, Treatment, and Overflow Model (STORM) of the Corps of Engineers Hydrologic Engineering Center was originally developed by Water Resources Engineers, Inc., and has since been revised and expanded a number of times. It was designed primarily to evaluate the stormwater storage and treatment capacities required to reduce untreated stormwater and combined sewage overflows below specified values [19, 20]. The model continuously simulates the time varying flow of wastewater and five conservative quality constituents from many urban and nonurban land uses within a single catchment, without routing them through a sewer network. It does not include treatment, or costs, and is not intended for design purposes. Data requirements are much the same as for the Simplified SWMM above, but with the additional need for gutter densities, depression storage capacities, accumulation rates by land use types and quality composition of dust and dirt, and street sweeping frequency and efficiency.

Hydrocomp Simulation Program--

The Hydrocomp Simulation Program (HSP) [21, 22], developed by Hydrocomp International, Inc., is the commercial successor to the Stanford Watershed Model. It is a very comprehensive mathematical model for simulating watershed hydrology and flow routing, which has been used extensively to model existing and planned surface water systems of both rural and urban catchments. It can be used for both single event and continuous wastewater flow and quality analysis (17 constituents). Of more concern to single event modeling, it does not include sedimentation and scour and water quality reactions in sewers, noncircular closed conduits, surcharging and pressure flow, sewer flow control and diversion structures, wastewater treatment, nor capabilities for design or cost calculations. HSP is a proprietary model, and users must contract with Hydrocomp in order to make routine applications; user's manuals are available describing the operation of the model. HSP data requirements are extensive and include: precipitation (rain and snow), air temperature, and streamflow; potential evapotranspiration, dew point, solar radiation, and wind velocity if available; and a number of calibration parameters (e.g., infiltration, depression storage, soil moisture storage, snow parameters, channel characteristics, watershed segments, and channel reaches) that require considerable user skill and experience to determine.

The MIT Urban Watershed Model--

The Massachusetts Institution of Technology Urban Watershed Model, or Catchment Model (MITCAT), was originally developed at MIT and later modified by Resource Analysis, Inc., for the U.S. Office of Water Resources Research [23, 24]. It simulates the time-varying runoff from several catchments and a sewer and open channel network including loops and coverging and diverging branches; it is limited to the simulation of single runoff events. MITCAT includes backwater and surcharging, and a separate model with design features to compute the sizes and costs of sewers, storage and treatment facilities which will result in the least cost combination of alternatives for the elimination of untreated overflows and the reduction of flooding and surcharging. The model does not include water quality or flow reversals. Potential users must contract with Resources Analysis, Inc., for routine applications. Detailed data are required on rainfall, the catchments, and streams; data are also required on evaporation, inflows, the storage reservoir, and soil erosion parameters.

SOGREAH Looped Sewer Model--

The Looped Sewer Model (CAREDAS), consisting of about 3000 Fortran Statements, was developed by the French consulting firm Societe Grenobloise d'Etudes et d'Applications Hydrauliques (SOGREAH) [25]. It is a very comprehensive model for routing wet- and dry weather flows and water quality constituents through complex sewerage systems, and appears to be limited to the simulation of single runoff events. It would be most useful where the consideration of backwater, downstream flow control, diversion structures, retention basins, surcharging, and flow reversal are important features of the sewerage system assessment. Multiple-shaped conduits and channels, inverted siphons, pumping stations, and receiving water flow and quality (separate program) are included. It does not include land use characteristics, snowmelt, sedimentation and scour, quality reactions, or design and costing capabilities; storage facilities appear to be included. CAREDAS is a proprietary model of SOGREAH, who have North American representatives in New York City and Lasalle, Quebec, Canada; a user's manual is available in French. Design hyetographs and dry-weather flow hydrographs are required for each inlet, rather than inputting recorded rainfall data. Catchment characteristics must be specified in moderate detail, and data on all the system components are required.

Urban Wastewater Management Model--

The Urban Wastewater Management Model (BNW), developed by Battelle-Northwest, is a comprehensive mathematical model for design optimization and/or real-time control considering wastewater quality objectives [26, 27]. It simulates the time-varying flows of wastewater and sewer quality constituents through major sewer system components, such as trunk and interceptor sewers, regulators, storage facilities, and treatment plants, for individual storm events only. It enables the performance of a planned or existing sewerage system to be evaluated under a variety of rainfall conditions without simulating all the minor system features, through the use of hydrologic lumping techniques. The required operation of control regulators during storms can be modeled in real time to minimize overflows. It is particularly suited to simulating large metropolitan areas without the need for defining each small sewer. BNW will also compute sizes and costs of structural modifications to major sewer system components, which will result in the least-cost combination of alternatives for improving system performance. The model does not include downstream flow control and flow reversal, in-line storage, land use considerations, wastewater quality reactions and sedimentation and scour, or receiving water flow and quality. Information on the general characteristics of the catchments is required in moderate detail; cumulative rainfall data per catchment, and the characteristics of all the system components are also required.

Storm Water Management Model--

The Storm Water Management Model (SWMM) was originally developed by Metcalf & Eddy, Inc., the University of Florida, and Water Resources Engineers, under the sponsorship of the EPA [7], and subsequently improved by the University of Florida [28]. This program, consisting of some 12 000 statements, is one of the most detailed and comprehensive mathematical models available for the simulation of single event urban stormwater runoff, storm sewer discharge, and combined sewer overflow phenomena. It includes eight water quality constituents, pumping stations and diversion structures, sedimentation and scour, treatment and storage (in-line and overflow), both with costs, and two-dimensional receiving water quality and quantity simulation. It does not include snowmelt, downstream flow control, or quality reactions in sewers and storage. The extensive SWMM data requirements are summarized in Table 82 of reference [8]. They include detailed data on the catchments and the system structures, information to define the system maintenance, the dry-weather flows, and the receiving waters, and numerous rainfall records, runoff hydrographs, and combined flow quality measurements.

The Hydrograph-Volume Method and Quantity-Quality Simulation Program--

These two models, HVM and QQS, were developed in Germany by Dorsch Consult [29, 30]. HVM is a single event model which does not simulate water quality. The QQS model, consisting of nearly 30 000 Fortran Statements, may be used for both continuous or single event quantity and quality simulation. Its extensive capabilities include backwater and surcharging, pumping stations and various diversion structures, storage, overflow and in-line treatment, and statistical analysis of results. It does not include snowmelt, sedimentation and scour, water quality reactions, or design and costing capabilities. The models are both proprietary, and arrangements to use them must be made with Dorsch Consult Ingenieurgesellschaft mbH of Munich, Germany; a user's manual is available. The detailed data requirements are generally similar to those of SWMM, above.

Water Resources Engineers Storm Water Management Model--

The Water Resources Engineers Storm Water Management Model (STORMSEWER) [31] is a modified version of SWMM, described previously. It is a single-event model noted for its capability to simulate all important hydraulic sewer phenomena, plus six water quality constituents. It should therefore be used in applications where the less common hydraulic phenomena are important. Major differences from SWMM are that it includes no in-line storage, treatment, or costing and design capabilities. Receiving water flow and quality can be simulated by a separate model. Its extensive data requirements are very similar to those of SWMM, however, with the additional requirement of ground surface elevations at manholes. This modeling option is available in SWMM releases after November 1975 under the identification "Extended Transport."

Seattle Computer Augmented Treatment and Disposal System--

The Seattle Computer Augmented Treatment and Disposal System (CATAD) of the Municipality of Metropolitan Seattle is an operating system for the real-time control of untreated overflows from the main trunk and interceptor sewers of the metropolitan Seattle, Washington, combined sewer system [32]. It does not include a comprehensive mathematical model for the simulation of combined wastewater runoff at this time; instead, it includes real-time data acquisition of rainfall, water level and quality data, and remote automatic computer control of regulators and pumping stations governed by rule curves and provided with manual overrides. Storage reservoirs, in-system water quality, and design and costing capabilities are not included. The programs are system specific and not transferrable for general application elsewhere. The system has been very effective in reducing pollutant discharges; however, a more comprehensive simulation model should be considered for new systems [15].

Selection of Mathematical Models

The key criterion in the selection of models is the problem under study. Planning projects will probably be viewing broad and significantly different development scenarios, perhaps basin-wide in scope; design projects will be more closely allied to casting a solution in concrete; and the designers of operational control systems will be most concerned with rapid response estimation and decision making in real time. Obviously, the precision required, range of alternatives to be modeled, and time available all differ greatly for each of these types of objectives. Numerous cases have occurred where a model was selected first, and then an entire study structured around the model; it is most important for study personnel to recognize that the model is simply a tool to assist in the conduct of their study.

Initially, the following questions should be addressed when considering the possibility of using models:

1. What is the problem to be solved?
2. What resolution is required?
3. Is a model needed, and if so, what approach is necessary?
4. What calibration and verification input data are available?
5. What are the client's requirements?

Each of these questions will be discussed.

The different broad types of problems which may be confronted have just been mentioned. Clearly, study objectives should be identified as specifically as possible, to facilitate communication, focus of effort, and commitment. These objectives should include statements of the study's major considerations, the controlling phenomena, the nature of the occurrence that is to be abated, and the types, locations, and performance of the controls. These will all help to identify model characteristics that may be needed (Table 8). Also desirable will be information on any unused capacity available, implementation and funding constraints, and the balance between known data and assumptions, and how this balance might change in the future.

The temporal resolution required may be by minute, hour, day, season, or even by year; it will depend strongly upon the type of water quality problem, hydraulic characteristics, and receiving water. Spatially, the resolution will depend upon the positions of points in the system where monitoring information is available and/or would be most beneficial. The selected model must be sufficiently sensitive to yield output differences within the known ranges of given variables.

If it has been established that the problem confronted is sufficiently complex that it justifies some kind of methodical approach, then models may be appropriate. However, these may range all the way from specific hand calculation procedures, through charts and nomographs, to computer models. Combinations of these various methods may also be used. Usually the simplest model which simulates the desired phenomena with adequately accurate mathematical formulations should be selected. Advanced models normally should not be used until their need has been demonstrated by less advanced ones; such simple demonstrations should also provide useful insight into the advanced study. Input data requirements and computer running times generally increase with increasing complexity of the model.

Desktop procedures may be entirely adequate where a gross assessment is required of the relative loads of various sources and their impacts on water quality. Continuous, simplified models add the benefits of total system perspective for problem identification and assessment for advanced planning, while detailed models enable final design evaluations and post construction assessments.

Some models include options to suppress portions of the simulation if only selected phenomena are of interest; such features should be considered in model selection. For some purposes, primarily the design the sanitary sewage systems, steady-state models are adequate to compute the least-cost combinations of sewer pipes and slopes for specified inflows. Nonsteady-state models are required, however, to adequately analyze complex storm and combined sewerage systems under dynamic runoff conditions. The simulation of quality adds considerable model complexity, even if limited to conservative substances. The complexity increases substantially if both storm- and dry-weather water quality are computed, and if wastewater treatment and receiving water flow and quality are being modeled.

Computer models follow operating rules without exception or interpretation, and can process mountains of data with relative ease. The developer sets the rules, and in most cases the options, and the user furnishes the data and is left with the results. In deciding how to model, and even whether or not to model, the prospective user should assess this process closely. What data does he have or can he get? What are the operating rules he can live with? What will he do with the results?

Models require input data upon which to operate, whether real or fabricated. The size and quality of the data base are likely to grow during the course of the work. To select models effectively requires that the data base strengths, weaknesses, and dynamics be identified beforehand. If suitable data are not available, and adequate funds and time for data collection are not provided, the use of a complicated model may be ruled out.

The results of the study for which models are contemplated must be approved by the client. His requirements and needs, including milestones and deliverables, and his budget resources, may have a strong influence upon whether and what type of models will be suitable for the job.

If, after preliminary analysis, it seems that a model is needed, several further factors should be considered:

1. Are suitably qualified personnel available to do the work? Modeling is an art, and should be carried out only under the direct supervision of suitably qualified and experienced professionals.

2. Are models available that have already been calibrated and applied locally? Since the final testing of alternatives is a relatively minor portion of the total cost of model application, a significant search effort is justified.

When the decision has been made to use models, a careful and systematic examination of available tools is warranted. Three reports published recently by the EPA provide invaluable information and guidance for this undertaking:

1. "Evaluation of Water Quality Models: A Management Guide for Planners," by Systems Control, Inc. (SCI) [33]

2. "Assessment of Mathematical Models for Storm and Combined Sewer Management," by Battelle-Northwest [14]

3. Appendix A: Model Applicability Summary, of "Areawide Assessment Procedures Manual," by the EPA Municipal Environmental Research Laboratory (MERL) [5]

The SCI report provides an excellent procedure for model evaluation and selection. Although this procedure is directed primarily at water quality models, the same general methodology could be effectively applied to drainage models too. The report also contains a chapter on contracting for modeling services which is applicable to any modeling activity. The model selection procedure is summarized in a recent technical paper [34]. The main objective of this procedure, which involves model performance index determinations and overall cost-effectiveness comparisons, is to provide a formal method, as objective as possible, for making comparisons. The selection procedure is organized into phases of increasing level of detail, each of which may or may not be required depending upon the nature of the problem being confronted.

The Battelle and MERL reports [5, 14], together with a subsequent technical paper [15], summarize in considerable detail most of the model characteristics required by the SCI selection procedure. Furthermore, the MERL document provides a model applicability summary prepared especially for the 208 planning process, which expands on many of the aspects reviewed herein.

When making model selections, in the manner described above, it would be well to keep in mind the following. All of the models are in use and undergoing continual change, so that the available descriptions may not be up-to-date. Also, most of the models have one or more derivatives, which may or may not be of equal or better utility.

Application of Mathematical Models

Model application may be thought of as consisting of three steps: calibration, verification, and analysis.

The purpose of calibration and verification is to adjust the values of all the unknown or uncertain model parameters until the model predictions correspond acceptably with the observed prototype behavior. Calibration is the first phase of parameter adjustment, with a first set of prototype input and output data for a representative simulation period. Verification then involves modeling at least one different simulation period, with a different set of prototype input and output data, using the originally calibrated parameters. If the model predictions for these subsequent simulation periods do not also agree acceptably with the corresponding prototype behavior, then the model is not yet verified, and further calibration, or recalibration, is necessary. While simulation periods for event models will contain selected storms, for continuous simulation models they should consist of suitably long periods of record. It is important that (1) the various simulation periods used for calibration and verification should correspond to significantly different prototype conditions, and that (2) some of them should approximate closely the conditions to be studied in the subsequent analysis.

The models described previously have generally demonstrated an ability to predict runoff quantities with sufficient accuracy and with relatively minor calibration effort required. This is not true of water quality, however. The models are very sensitive to pollutant accumulation and washoff functions, and as a result most of the model calibration effort needs to be directed to this area. In particular, any default values for calibration parameters should be carefully examined for their appropriateness to the local situation. While runoff quantity may be satisfactorily calibrated using streamflow data, runoff quality should be calibrated using current local information. One technique for facilitating applications of detailed models to extensive areas is to calibrate them on one or more small subcatchments, and then extrapolate the results to the larger study area.

Very large amounts of data may be required for calibration and verification. These will include rainfall, catchment characteristics, sewer network, sewer flow, and receiving water data. Generally, the most prized data represent cause-effect relationships. Examples might include changes in plant inflow characteristics between dry- and wet-weather periods; measured rainfall-runoff-quality relationships; intensive stream surveys correlated with development, treatment performances, and, ideally, storm occurrences; and documentation as to the time, location, and duration of overflows.

More sophisticated models generally require proportionately more input data. Although the gathering of the mass of data required for a very comprehensive model can be a formidable task, it usually comprises a major share of the cost of modeling, and the subsequent use of the model for evaluating proposed additions or modifications to a sewerage system can contribute to large cost savings. Very helpful information on monitoring and sampling for data collection is provided in the Envirex report [10] discussed in the preceding review of planning guides.

With calibration and verification satisfactorily completed, the model is finally ready to be used for analysis. This involves running the model for various test cases for which adequate prototype output data cannot be obtained economically. All such model runs should be made bearing in mind the extent to which the simulated conditions have been extrapolated from the calibration and verification conditions. The analysis exercises should be relatively straightforward, unless the results indicate, as they often will, that unexpected conditions have been met and further assessment or calibration is needed.

The utility of model applications is illustrated in the following subsection.

ILLUSTRATIVE PROBLEMS

The following examples illustrate different model applications to a single watershed. The models chosen for these demonstrations range from desktop models (Level I: problem assessment) through continuous simulation models (Level II: planning) to single event simulation models (Level III: analysis). The examples were chosen to illustrate alternative approaches to solving a variety of urban runoff pollution problems.

The basin (typically California suburban, medium density residential) is first broadly characterized, for the benefit of all examples, by the general characteristics listed in Table 9. The basin is shown in Figure 13. Each subsequent example problem includes a statement of its objectives, additional specific conditions which apply, assumptions made, the solution in a series of steps, and concluding comments. The objective of these examples, particularly for the more complex computer models, is not to provide reproducible details of the computations, but rather to illustrate the varied capabilities of the approach methodologies.

TABLE 9. DEMONSTRATION AREA GENERAL CHARACTERISTICS

1.	Watershed name	Calabazas Creek
2.	Watershed location	Santa Clara County, California
3.	Total area, acres	9070
4.	Total population	72 947
5.	Mean annual precipitation, in.	18.35
6.	Average street sweeping interval, N_s, days	14 (1 pass each 2 wk)
7.	Average BOD generation rate, lb/capita·d	0.24
8.	Average SS generation rate, lb/capita·d	0.24
9.	Residential population by sewer type	
	Combined	0
	Storm	72 947
	Unsewered	0
	Total	72 947
10.	Land use by sewer type, acres	

	Land use							
System type	Residential	Residential multifamily	Commercial	Industrial	Developed open space	Undeveloped open space	Agricultural	Total
Undeveloped	0	0	0	0	0	3114	0	3114
Combined	0	0	0	0	0	0	0	0
Storm	3431	0	1333	154	1038	0	0	5956
Unsewered developed	0	0	0	0	0	0	0	0
Total	3431	0	1333	154	1038	3114	0	9070

acre x 0.405 = ha
in. x 2.54 = cm
lb x 0.454 = kg

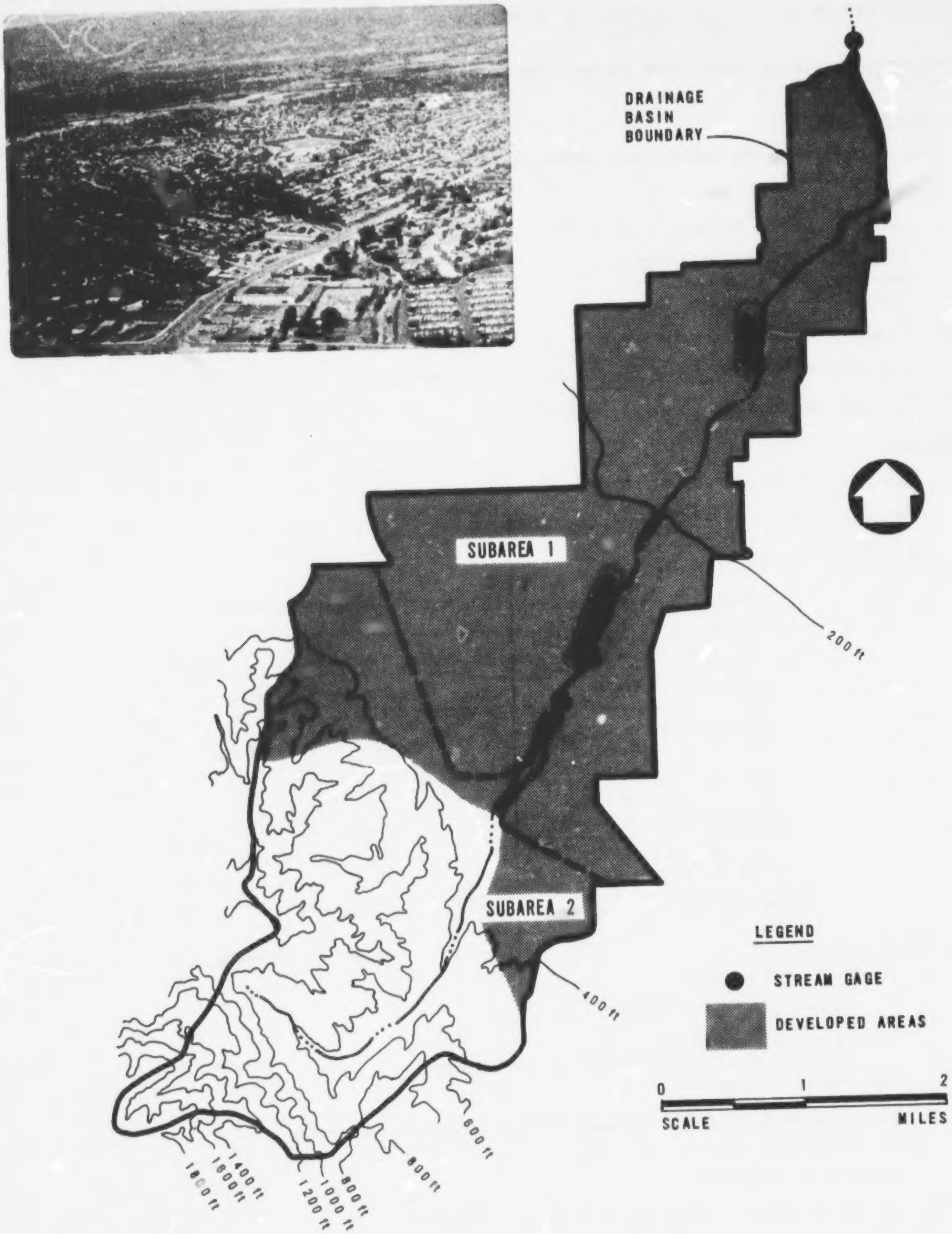

DRAINAGE BASIN BOUNDARY
SUBAREA 1
SUBAREA 2
200 ft
400 ft
600 ft
800 ft
800 ft
1000 ft
1200 ft
1400 ft
1600 ft
1800 ft
LEGEND
● STREAM GAGE
▨ DEVELOPED AREAS
0
1
2
SCALE
MILES

FIGURE 13. The Calabazas Creek drainage basin.

EXAMPLE PROBLEM 4-1: INITIAL ASSESSMENT OF THE MANAGEMENT PROBLEM (UNIVERSITY OF FLORIDA DESKTOP "LEVEL I" METHOD [3])

Determine the average annual flows and pollutant loads in the wet-weather and dry-weather flows from the specified area.

Specified Conditions

(None in addition to the General Characteristics, Table 9)

Assumptions

1. Depression storage by land use [3, p 13] is:

Land use	Depression storage, in.
Impervious	0.0625
Pervious	0.25

2. Dry-weather flows are generated only by residential land use.
3. Population resides entirely within residential land use areas.
4. The total population is served by sanitary sewers.

Solution

1. Population density by sewer system type.

$$\text{Population density, PD} = \frac{\text{population (Table 9, Item 9)}}{\text{area, acres (Table 9, Item 10)}}$$

See results in Table 4-1.

TABLE 4-1. RUNOFF ANALYSIS RESULTS FOR EXAMPLE PROBLEM 4-1

Step	Item	Undeveloped	Combined	Storm	Unsewered developed	Total	Average
	Population (Table 9, Item 9)	0	0	72 947	0	72 947	.....
	Area, acres (Table 9, Item 10)	3 114	0	5 956	0	9 070	.....
1	Population density, PD, people/acre	0	..	12.25	..	......	12.25[a]
2a	Imperviousness, I, %	0	..	36.3	..	......	.....
2b	Runoff coefficient, CR	0.150	..	0.422	..	......	.....
2c	Depression storage, DS, in.	0.25	..	0.18	..	......	.....
	Annual precipitation, P, in./yr (Table 9, Item 5)	18.35	..	18.35	..	......	18.35
2d	Annual runoff, AR, in./yr	0.46	..	5.85	..	......	4.00
2e	Dry-weather flow, DWF, in./yr	......	..	16.42	..	......	10.78

a. For developed areas only.

2. Runoff analysis
   a. Percent imperviousness

   $$I = 9.6\ PD^{(0.573\ -\ 0.0391\ \log_{10} PD)} \quad [3,\ p\ 10]$$

   where I = imperviousness, %
   PD = population density, people/acre (from Step 1).

   See results in Table 4-1.

   b. Runoff coefficient (weighted between pervious and impervious areas).

   $$CR = 0.15 + 0.75\ (I/100) \quad [3,\ p\ 10]$$

   See results in Table 4-1.

   c. Depression storage (area weighted; based on Assumption 1).

   $$DS = 0.25 - 0.1875\ (I/100),\ \text{in.} \quad [3,\ p\ 13]$$

   See results in Table 4-1.

d. Annual runoff

$$AR = (CR)P - 5.234(DS)^{0.5957}, \text{ in./yr} \quad [3, \text{p } 13]$$

where P = annual precipitation, in./y (Table 9, Item 5)

See results in Table 4-1.

e. Dry-weather flow

$$DWF = 1.34\ PD_d\ , \text{ in./yr} \quad [3, \text{p } 13]$$

where PD_d = average population density, people/acre, for all developed areas only (Table 4-1, Step 1)

See results in Table 4-1.

3. Quality analysis

a. Street sweeping effectiveness factor, γ

$$\gamma = \begin{cases} N_s/20, & 0 \le N_s \le 20 \text{ days} \\ 1.0, & N_s > 20 \text{ days} \end{cases} \quad [3, \text{Table 8, p } 17]$$

where N_s = street sweeping interval, days

See results in Table 4-2.

TABLE 4-2. QUALITY ANALYSIS RESULTS FOR EXAMPLE PROBLEM 4-1

		Land use					
Step	Item	Residential	Commercial	Industrial	Other developed	Undeveloped	All
..	Area, A, acres (Table 9, Item 10)	3 431	1 333	154	1 038	3 114	9 070
..	Street sweeping interval, N_s, days (Table 9, Item 6)	14	14	14	14	.....	.....
3a	Street sweeping effectiveness factor, γ	0.7	0.7	0.7	0.7	.....	.....
..	Developed population density, PD_d, people/acre (Table 4-1)	12.25[a]	12.25[a]	12.25[a]	12.25[a]	0	.....
3b	Population density function, f_2 [3, Table 8]	0.985	1.0	1.0	0.142	.....	.....
..	Annual precipitation, P, in./yr (Table 9, Item 5)	18.35	18.35	18.35	18.35	18.35	18.35
3c	Average annual wet-weather loading rate for separate sewer areas, M, lb pollutant/acre·yr:						
	BOD_5	10.1	41.1	15.5	0.2	0	10.2[b]
	SS	206	285	374	5	0	127[b]
	TVS	120	180	184	5	0	75[b]
	$T\text{-}PO_4$	0.425	0.972	0.906	0.018	0	0.321[b]
	T-N	1.66	3.80	3.56	0.11	0	1.26[b]
..	Average BOD generation rate, lb/capita·d (Table 9, Item 7)	0.24	......	....	....	.....	......
..	Population, N_p, (Table 9, Items 8 and 9)	72 947	0	0	0	0	72 947
3d	Dry-weather BOD loading, M_{DW}, lb BOD/acre·yr	1 862	0	0	0	0	704
3d	Dry-weather BOD after 85% treatment, lb BOD/acre·yr	279	0	0	0	0	106

a. Average for all developed areas only.

b. Average weighted on the basis of area.

b. Population density function, f_2

Land use	f_2 [3, Table 8]
Residential	$0.142 + 0.218\ (PD_d)^{0.54}$
Commercial, industrial	1.0
Other developed	0.142

where PD_d = average population density for all developed areas only, people/acre (Table 4-1, Step 1)

See results in Table 4-2.

c. Average annual wet-weather loading rates.

$M = \alpha P f_2 \gamma$ [3, p 17]

where M = lb pollutant/acre·yr
γ = factor for separate sewer areas [3, Table 8]
P = annual precipitation, in./yr

See results in Table 4-2.

d. Dry-weather flow BOD loadings

$M_{DW} = 365\ G\ N_p/A$ [3, p 20]

where M_{DW} = dry-weather BOD loading, lb/acre·yr
G = average BOD generation rate, lb/capita·d (Table 9, Item 7)
N_p = population (Table 9, Items 8 and 9)
A = populated area, acres (Table 9, Item 10)

Notes: (1) No correction for deposition in sewers included in this estimate.
(2) Based on assumptions 2 and 4.

See results in Table 4-2.

Comments

From Tables 4-1 and 4-2 we see that:

1. The average annual basin runoff coefficient is 0.22 (4.00 in./18.35 in.)
2. The annual wet-weather runoff from the total watershed is 37% (4.00 in./10.78 in.) of the annual dry-weather flow.
3. The probable magnitudes of the annual loads of significant wet-weather pollutants are listed under Step 3c of Table 4-2.
4. The annual wet-weather BOD load will be about 92 100 lb, of which 60% originates from commercial land use areas and 38% from residential area.
5. The average annual BOD concentration is 11.2 mg/L (from 10.2 lb BOD/acre·yr and 4.00 in. runoff/yr).
6. The annual wet-weather BOD load will be about 10% (10.2/106) of the annual dry-weather BOD load after 85% treatment.
7. Likewise, the annual wet-weather SS load will be about 1.2 (127/106) times the annual dry-weather SS load after 85% treatment.

EXAMPLE PROBLEM 4-2: INITIAL ASSESSMENT OF THE MANAGEMENT PROBLEM (URS DESKTOP METHOD [4])

Determine the time variation of wet-weather flow and quality at a basin outlet, resulting from a "design" storm on the basin.

Specified Conditions

	Subarea 1	Subarea 2
1. Size, acres	5 049	4 021
2. Area of streets, acres	773	160
3. Interconnected impervious area, acres	1 535	272
4. Length of main drainage channel, ft	33 000	14 300
5. Average slope of main drainage channel, ft/ft	0.0072	0.0217
6. Length of curbs, miles	277.2	57.4
7. Channel material	Natural, in moderate condition	

Assumptions

1. Storm starts 8 days after last substantial (greater than 0.5 in.) rainfall.
2. Storm starts 3 days after streets last swept.
3. Street sweeping efficiency = 40% (a representative average value).

4. Street surface solids loading rate and composition are the same as the nationwide mean, viz. [4, Table 3, p I-11):

Dry solids lb/curb mi·d	Concentration, micrograms/gram of dry solids	
	BOD5	TPO4
156	19 900	2 930

Note: The use of tables of values for specific area characteristics (Level II), or site specific data (Level III), is preferable whenever appropriate.

5. Equivalent curb miles per acre of street surface = 0.46 (average for all land use types) [4, Table 2, p I-10).
6. Runoff from pervious areas in negligible; therefore imperviousness of runoff generating surfaces is 100%.
7. A uniform runoff of 0.5 in./h will wash off 90% of the initial street surface pollutants in 1 hour.
8. All urban contaminants are assumed to derive from street surfaces only; they are removed by street surface runoff, and then mix with impervious runoff.
9. Outfall is located at outlet to Subarea 1.
10. Channel flow depth during design storm is 2.0 ft, in trapezoidal channel with 20 ft base width and 1 on 1.2 side slopes. Hydraulic radius therefore is 1.72 ft.

Solution

1. Determine equivalent days of solids accumulation.

$EDA = D_s + (D_r - D_s)(1 - E_s)$ [4, p I-8]

where EDA = equivalent days of accumulation
D_s = days since last swept
D_r = days since last substantial rainfall
E_s = street sweeping efficiency

From Assumptions 1 through 3:

$EDA = 3 + (8 - 3)(1 - 0.4) = 6.0$ days

2. Compute the dry solids load/curb mile at start of storm.

Solids load, L = rate of accumulation x EDA [4, p I-7]
= 156 lb/curb mi·d x 6 days (from Assumption 4 and Step 1)
= 936 lb/curb mi

3. Convert solids load to lb/acre of street surface.

lb solids/acre = lb/curb mi x curb mi/acre
= 936 x 0.46 (from Step 2 and Assumption 5)
= 431

Note: The contaminant compaction of the solids is provided by the data of Assumption 4.

4. Select a design storm for the study area and determine the resulting runoff rate and duration.

Following the guidance of [4, Section 3.3.2], select a 1 year, 30 minute storm. From the corresponding U.S. rainfall chart [4, Figure 4, p I-25], this is seen to contribute 0.5 in. of rainfall in the Santa Clara County area of California.

The runoff coefficient, k , for impervious surfaces is given in [4, Table 8, p I-19] for various slopes. For our watershed, then:

	Subarea 1	Subarea 2
Slope (from Table 9)	<2%	>2%, <7%
Approximate k of impervious surfaces [3, Table 8]	0.80	0.85

By the runoff coefficient method [4, p I-18]:

$R = kP$

where R = runoff rate, in./h
P = precipitation rate, in./h
k = runoff coefficient

	Subarea 1	Subarea 2
P, in./h	1.0	1.0
k	0.80	0.85
R = kP, in./h	0.80	0.85
R.O. duration	30 min	30 min

5. Synthesize unit hydrographs for the interconnected impervious portion and the street surface portion of each subarea, using Espey's equations [4, Figures 7-11]). Then modify the unit hydrographs to reflect the runoff rates from the design storm by multiplying the flows by the ratio, r,

$$\text{where } r = \frac{\text{design storm runoff rate, in./h}}{\text{unit hydrograph runoff rate, in./h}}$$

By definition, a unit hydrograph has 1 in. of direct runoff.

	Subarea 1	Subarea 2
Given:		
Impervious area, A_I, acres (specified condition)	1 535	272
Street surface area, A_S, acres (specified condition)	773	160
Imperviousness, % (Assumption 6)	100	100
Channel length, ft (specified condition)	33 000	14 300
Channel slope, ft/ft (specified condition)	0.0072	0.0217
Design storm runoff rate, in./h (Step 4)	0.80	0.85
Runoff duration, h (Step 4)	0.50	0.50
Unit hydrograph runoff rate, in./h	2.00	2.00
Determine:		
Q_{IUP}/A_I, ft^3/s·acre [4, Figure 7]	0.83	1.20
Q_{SUP}/A_S, ft^3/s·acre [4, Figure 7]	0.83	1.20
T_R, min [4, Figure 8]	45	34
T_B, min [4, Figure 9]	295	220
W_{50}, min [4, Figure 10]	67	48
W_{75}, min [4, Figure 11]	42	32
Q_{IUP}, ft^3/s (by mult.)	1 274	326
Q_{SUP}, ft^3/s (by mult.)	642	192
Ratio, r	0.4	0.425
Q_{IP}, ft^3/s (r Q_{IUP})	510	139
Q_{SP}, ft^3/s (r Q_{SUP})	256	82

Note: I = impervious areas; S = street surfaces only; U = unit hydrograph; P = peak flow.

Using the proportions of [4, Figure 6], the resultant design storm runoff hydrographs are shown in Figure 4-1.

6. Determine the time-varying quality of the runoff at each subarea outlet.

	Subarea 1	Subarea 2
Curb length, miles (specified conditions)	277.2	57.4
Initial solids load, lb/curb mi (Step 3)	431	431
∴ Initial solids load, lb	119 473	24 739
Street surface area, A_S, acres (specified condition)	773	160
Time step size, Δt, min (approximately $T_B/20$)	15	10
60.5 A_S Δt	701 498	96 800

Perform the pollutograph and loadograph calculations according to the URS method, using the following equations:

Incremental volume of street surface runoff, $\Delta V_S = \bar{Q}_S$ (ft^3/s) Δt (min) 60, ft^3

Average street surface runoff rate, $\bar{r} = \frac{\Delta V_S \text{ (ft}^3\text{)}}{60.5\ A_S \text{ (acres) } \Delta t \text{ (min)}}$, in./h

Surface solids load, $P_0 = P_0$ (previous) - ΔP, lb

Incremental solids removal, $\Delta P = 4.6\ \bar{r}$ (in./h) P_0 (lb) $\frac{\Delta t \text{ (min)}}{60}$, lb

Cumulative solids removed, $\Sigma\Delta P = \Sigma\Delta P$ (previous) + ΔP, lb

Incremental volume of impervious surface runoff, $\Delta V_i = \bar{Q}_i\ (\mathrm{ft}^3)\ \Delta t\ (\mathrm{min})\ 60$, ft^3

Instantaneous solids concentration, $C = \dfrac{\Delta P\ (\mathrm{lb})\ 16\ 019}{\Delta V_i\ (\mathrm{ft}^3)}$, mg/L

Pollutograph and loadograph worksheet for Subarea 1:

	Street surface runoff						
Δt, min	Volume, ΔV_s, ft^3	r_{avg}, in./h	P_o, lb	ΔP, lb	$\Sigma\Delta P$, lb	Total volume, ΔV_i, ft^3	Total solids concentration, mg/L
0-15	21 600	0.0308	119 473	4 232	4 232	22 000	3 081
15-30	129 600	0.1848	115 241	24 491	28 723	216 000	1 816
30-45	208 800	0.2976	90 750	31 058	59 781	439 200	1 133
45-60	230 400	0.3284	59 692	22 543	82 324	444 600	812
60-75	192 600	0.2746	37 149	11 731	94 055	369 000	509
75-90	136 800	0.1950	25 418	5 700	99 755	275 400	332
90-105	104 400	0.1488	19 718	3 374	103 129	183 600	294
105-120	68 400	0.0976	16 344	1 834	104 963	135 000	218
120-135	50 400	0.0718	14 510	1 198	106 161	97 200	197
135-150	39 600	0.0564	13 312	863	107 024	77 400	179
150-165	30 600	0.0436	12 449	624	107 648	59 400	168
165-180	23 400	0.0334	11 825	454	108 102	46 800	155
180-195	19 000	0.0270	11 371	353	108 455	36 000	157
195-210	13 600	0.0194	11 018	246	108 701	30 600	129
210-225	10 800	0.0154	10 772	191	108 892	24 400	125
225-240	8 200	0.0116	10 581	141	109 033	16 200	139
240-255	6 400	0.0092	10 440	110	109 143	12 600	140
255-270	4 600	0.0066	10 330	78	109 221	9 000	139
270-285	2 400	0.0034	10 252	40	109 261	5 400	119
285-300	600	0.0008	10 212	9	109 270	3 000	48

Note: Repeat these computations for Subarea 2 (see Step 7 for results).

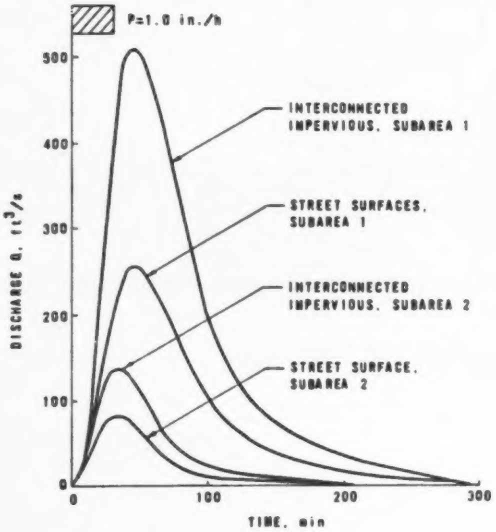


Figure 4-1. Design storm runoff hydrograph.

7. Route the quantity and quality graphs, to determine outfall conditions.

	Channel below	
	Subarea 1	Subarea 2
Channel material (specified condition)	..	Natural
Adjustment factor [4, Table 10]	..	2.3
Hydraulic radius (Assumption 10)	..	1.72
Channel slope, ft/ft (specified condition)	..	0.0072
Travel distance, ft (specified condition)	0	33 000
∴ lag time, min [4, Figure 18 and Table 10]	0	96

From [4, p I-59], the outlet time-varying flows and quality are given by:

Flow, $Q_{o,t} = Q_{1,t} + Q_{2,t-\Delta t}$

Concentration, $C_{o,t} = \dfrac{(Q_{1,t})(C_{1,t}) + (Q_{2,t-\Delta t})(C_{2,t-\Delta t})}{Q_{1,t} + Q_{2,t-\Delta t}}$

Load, $L_{o,t} = L_{1,t} + L_{2,t-\Delta t}$

where subscripts 1 and 2 indicate subarea outlet conditions and Δt = lag time from outlet 2 to 1.

Time, t, min	$Q_{1,t}$, ft^3/s^a	$Q_{2,t-\Delta t}$ ft^3/s^a	$Q_{o,t} = Q_{1,t} + Q_{2,t-\Delta t}$ ft^3/s	$C_{1,t}$ mg/L	$C_{2,t-\Delta t}$ mg/L	$C_{o,t}$ mg/L	$L_{1,t}$ lb	$L_{2,t-\Delta t}$ lb	$L_{o,t}$ lb
0	0	0	0	?	0	?	0	0	0
15	116	0	116	2 320	0	2 320	4 200	0	4 200
30	374	0	374	1 440	0	1 440	28 700	0	28 700
45	512	0	512	960	0	960	59 800	0	59 800
60	460	0	460	620	0	620	82 300	0	82 300
75	362	0	362	410	0	410	94 000	0	94 000
90	256	0	256	300	0	300	99 800	0	99 800
105	176	30	206	240	2 160	520	103 100	1 100	104 200
120	130	120	250	200	1 520	834	105 000	9 500	114 500
135	96	134	230	170	700	479	106 100	17 300	123 400
150	72	100	172	160	310	247	107 000	20 700	127 700
165	58	54	112	150	210	179	107 600	21 900	129 500
180	48	32	80	150	160	154	108 100	22 200	130 300
195	38	22	60	140	130	136	108 500	22 500	131 000
210	30	18	48	140	120	133	108 700	22 600	131 300
225	26	14	40	140	120	133	108 900	22 700	131 600
240	20	10	30	140	120	133	109 000	22 700	131 700
255	14	8	22	140	120	133	109 100	22 800	131 900
270	10	6	16	130	120	126	109 200	22 900	132 100
285	6	4	10	70	120	90	109 300	23 000	132 300
300	0	2	2	0	150	150	109 300	23 000	132 300
315	0	0	0	0	0	0	109 300	23 100	132 400
			Σ = 3 358						

a. Interconnected impermeable.

The outlet flows and quality are plotted in Figure 4-2. The concentrations (as shown) and cumulative loads of several pollutants may be easily displayed on the same figure, by adding ordinate scales based on the proportionalities (to total solids) of Assumption 4. For example, BOD load equals 0.0199 times total solids load.

The total runoff from the storm may be computed from the area under the hydrograph (Figure 4-2 and Step 7 tabulations) as follows:

$$R = \frac{\Sigma Q_{o,t}\ (\text{ft}^3/\text{s})\ \Delta t\ (\text{min})\ 60\ (\text{s/min})\ 12\ (\text{in./ft})}{9070\ (\text{acre})\ 43\ 560\ (\text{ft}^2/\text{acre})},\ \text{in.}$$

$$= \frac{3358(15)60(12)}{9070\ (43\ 560)}$$

$$= 0.092\ \text{in.}$$

Comments

1. The total runoff (area under hydrograph) is 0.09 in. This is 18% of the total storm precipitation (0.50 in.). The peak discharge is 512 ft^3/s.

2. Total BOD washoff by this analysis is 132 400 x 0.0199 = 2635 lb

3. **The average BOD concentration for the storm is 13.9 mg/L (from 2635 lb BOD and 0.092 in. runoff).**
4. **Most of the pollutant load (128 000 lb total solids) is released by 150 min after the start of the storm. By that time the BOD concentration (maximum over 45 mg/L) in the outflow is less than 5 mg/L.**

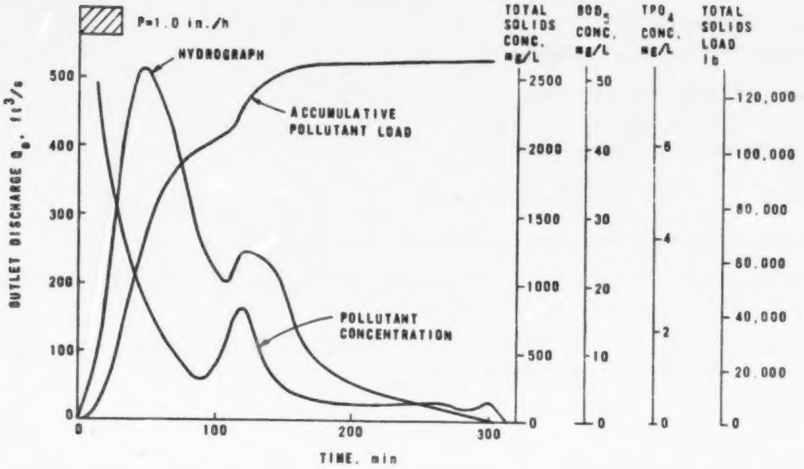


Figure 4-2. Quantity/quality graphs reflecting several parameters at the basin outlet.

EXAMPLE PROBLEM 4-3: PRELIMINARY PLANNING OF INTEGRATED STORAGE AND TREATMENT (UNIVERSITY OF FLORIDA DESKTOP "LEVEL I" METHOD [3])

Determine

1. The optimum wet-weather storage and treatment capacities and costs required to achieve 75% BOD removal;
2. Whether secondary wet-weather treatment is more cost effective than tertiary dry-weather treatment in this basin; and
3. The reduction in annual costs of managing wet-weather flows made possible by the following methods of integrating pollution control facilities:
    - Diverting some of the wet-weather flows to the dry-weather treatment plant for treatment in excess capacity available there
    - Performing some wet-weather flow treatment (quality control) within storage provided for wet-weather flow quantity control

Specified Conditions

(None in addition to the General Characteristics, Table 9).

Note: This problem builds on Example Problem 4-1, since both follow the same University of Florida guide [3].

Assumptions

1. Following the University of Florida procedure, the annual runoff derived earlier including that for depression storage ([3, p 13] and Example Problem 4-1, Step 2d) is not used for this analysis. Instead, the annual runoff from five regional reference cities was determined from computer runs with the STORM model [3, p 33]. The average runoff relation for these five cities, AR = 0.974(CR)P, is therefore used in Solution Step 1 below.

2. Representative annual (debt service plus operation and maintenance) storage and treatment costs are [3, p 30]:

   - Storage: $122e^{0.16(PD)}$, \$/acre-in.
     where PD = gross population density, people/acre
   - Primary treatment: \$4000/Mgal·d
   - Secondary treatment: \$15 000/Mgal·d

3. Representative treatment efficiencies are [3, p 26]

Treatment device	Assumed efficiency, η (BOD_5 removal)
Primary	0.40
Secondary	0.85

4. The average dry-weather flow generation rate is 100 gal/capita·d

5. A tertiary dry-weather treatment plant increases the BOD removal efficiency from $\eta_{sec} = 0.85$ to $\eta_{tert} = 0.95$

6. Excess capacity (E>0) equal to the average daily flow exists in the sewage treatment plant (for Cases (b) and (d) of Figure 4-6).

Solution

1. Compare the characteristics of the study area (Calabazas Creek drainage basin) with those of the regional reference city.

   For the Calabazas Creek basin:

   Study area is in Region I [3, p 47]
   Reference city in Region I is San Francisco [3, p 23]

   Total population = 72 947 (Table 9, Item 9)

   Total basin area = 9070 acres (Table 9, Item 10)

   Gross population density, $PD = \frac{72\ 947}{9\ 070} = 8.05$ people/acre

   Gross imperviousness, $I = 9.6\ PD^{(0.573 - 0.0391 \log_{10} PD)}$ [3, p 10]
   $= 29.4\%$

   Gross runoff coefficient, $CR = 0.15 + 0.75\ (I/100)$ [3, p 10]
   $= 0.371$

   Annual rainfall, P = 18.35 in. (Table 9, Item 5)

   Annual runoff, $AR = 0.974(CR)P$ (Assumption 1)
   = 6.63 in.

	San Francisco (r = reference city) [3, p 33]	Calabazas Creek (s = study area)
Imperviousness, I, %	32.9	29.4
Runoff coefficient, CR	0.397	0.371
Annual rainfall, P, in.	24.26	18.35
Annual runoff, AR, in.	9.37	6.63

2. Determine isoquant coefficients a, b, h, d, and f for the study area (subscript s) from those for the reference city (subscript r) using the following adjustment equations [3, p 48]:

$$a_s = \frac{AR_s}{8.76 \times 10^5}\ ;\ b_s = \frac{AR_s}{AR_r} b_r\ ;\ h_s = h_r\ ;\ d_s = \frac{AR_r}{AR_s} d_r\ ;\ f_s = f_r$$

Isoquant coefficient	San Francisco (r = reference city) [3, p 41]	Calabazas Creek (s = study area) (by adjustment)
a, in./h·(% R)	0.000 010 7	0.000 007 57
b, in./h	0.002 165	0.001 531
h, /(% R)	0.038 84	0.038 84
d, /in.	211.3	298.6
f, /(% R)	0.032 02	0.032 02

where R is the percent runoff control (treatment).

Note: Given these coefficients and the general isoquant equation [3, p 16], isoquant curves similar to Figure 10 may be plotted, enabling a graphical determination of the optimum storage and treatment capacities. The alternate, numerical procedure is followed below, however.

3. Determine constant unit costs of storage and treatment.

Gross population density, PD = 8.05 people/acre (Step 1)

Annual storage cost = $122e^{0.16(PD)}$, \$/acre·in. (Assumption 2)
= $122e^{0.16(8.05)}$
= 441.61 \$/acre·in.
= 0.01626 \$/gal

Annual primary treatment cost = \$4000/Mgal·d (Assumption 2)
= \$2608/acre-in.·h

Annual secondary treatment cost = \$15 000/Mgal·d (Assumption 2)
= \$9780/acre-in.·h

4. Calculate optimal storage and treatment capacities for the basin, for various degrees of pollutant control, following the University of Florida procedure of [3, p 51]. This makes use of the isoquant coefficients of Step 2, the unit costs of Step 3, and the treatment efficiencies of Assumption 3.

Type of control	Runoff control, R %	$K = de^{-fR}$ in.$^{-1}$	$T_2-T_1 = be^{hR}$ in./h	$T_1 = aR$ in./h	$T_2 = (T_2-T_1)+T_1$ in./h	Annual unit costs: Storage C_S \$/acre-in.	Annual unit costs: Treatment C_T \$/acre-in.·h	$\frac{C_T}{C_S}$ h	Optimal storage, S^{*a} in.	Optimal treatment, T^{*b} in./h	Annual cost of optimal control Z^{*c} \$/acre	Efficiency of unit η	BOD removal, $R_1 = \eta R$ %
Primary	0	298.6	0.00153	0	0.00153	441.61	2608	5.906	0.0033	0.00057	2.95	0.4	0
	10	216.8	0.00242	0.00008	0.00249	441.61	2608	5.906	0.0052	0.00086	4.54	0.4	4
	25	134.1	0.00404	0.00019	0.00423	441.61	2608	5.906	0.0087	0.00145	7.61	0.4	10
	50	60.23	0.01068	0.00040	0.01106	441.61	2608	5.906	0.0222	0.00319	18.11	0.4	20
	75	27.05	0.02819	0.00057	0.02876	441.61	2608	5.906	0.0556	0.00683	42.23	0.4	30
	90	16.73	0.05048	0.00068	0.05116	441.61	2608	5.906	0.0961	0.01080	70.58	0.4	36
	100	12.15	0.07443	0.00076	0.07519	441.61	2608	5.906	0.1379	0.01470	99.23	0.4	40
Secondary	0	298.6	0.00153	0	0.00153	441.61	9780	22.15	0.0078	0.00015	4.90	0.85	0
	10	216.8	0.00242	0.00008	0.00249	441.61	9780	22.15	0.0113	0.00028	7.77	0.85	8.5
	25	134.1	0.00404	0.00019	0.00423	441.61	9780	22.15	0.0185	0.00053	13.33	0.85	21.3
	50	60.23	0.01068	0.00040	0.01106	441.61	9780	22.15	0.0441	0.00113	30.52	0.85	42.5
	75	27.05	0.02819	0.00057	0.02876	441.61	9780	22.15	0.1045	0.00224	68.03	0.85	63.8
	90	16.73	0.05048	0.00068	0.05116	441.61	9780	22.15	0.1751	0.00338	110.35	0.85	76.5
	100	12.15	0.07443	0.00076	0.07519	441.61	9780	22.15	0.2467	0.00447	152.71	0.85	85.0

---

a. $S^* = \max\left[\frac{1}{K}\ln\left\{\frac{C_T}{C_S}K(T_2-T_1)\right\}, 0\right]$

b. $T^* = T_1 + (T_2-T_1)e^{-KS^*}$

c. $Z^* = C_S(S^*) + C_T(T^*)$

[3, p 51]

Convert the above results to convenient units:

Type of control	Runoff control, R %	T_2 Mgal/d	Optimal storage, S* Mgal	Optimal treatment, T* Mgal/d	Optimal annual cost, $1000 Storage	Treatment	Total	BOD removal, R_1 %
Primary	0	9.1	0.8	3.4	13	14	27	0
	10	14.7	1.2	5.1	20	20	40	4
	25	25.0	2.1	8.6	34	34	69	10
	50	65.4	5.5	18.9	89	75	164	20
	75	170.1	13.7	40.4	222	161	383	30
	90	302.5	23.7	63.9	385	255	640	36
	100	444.7	34.0	86.9	553	348	901	40
Secondary	0	9.1	1.9	0.9	31	13	44	0
	10	14.7	2.8	1.7	46	26	72	8.5
	25	25.0	4.6	3.1	74	47	121	21.3
	50	65.4	10.9	6.7	177	100	277	42.5
	75	170.1	25.8	13.2	419	198	617	63.8
	90	302.5	43.1	20.0	701	300	1001	76.5
	100	444.7	60.8	26.5	989	397	1386	85.0

The resulting capacity and cost relationships are presented graphically in Figures 4-3 and 4-4, respectively.

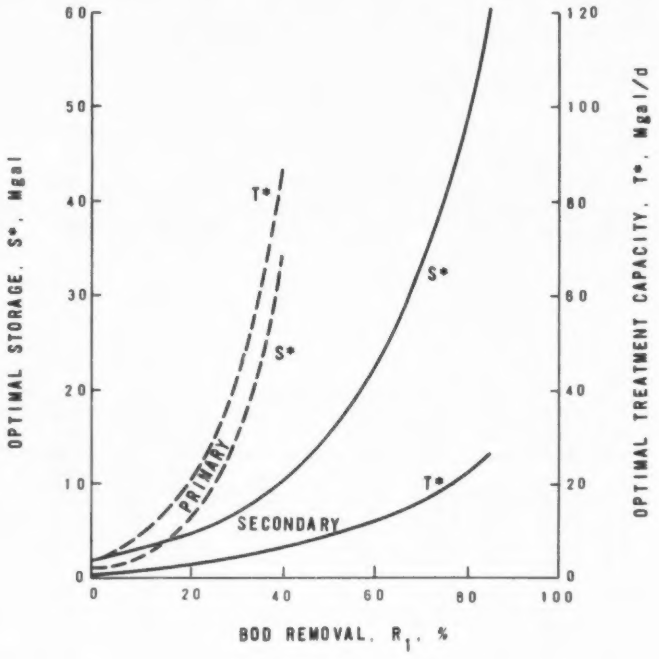


Figure 4-3. Variation of optimal storage and treatment capacities with degree of BOD removal.

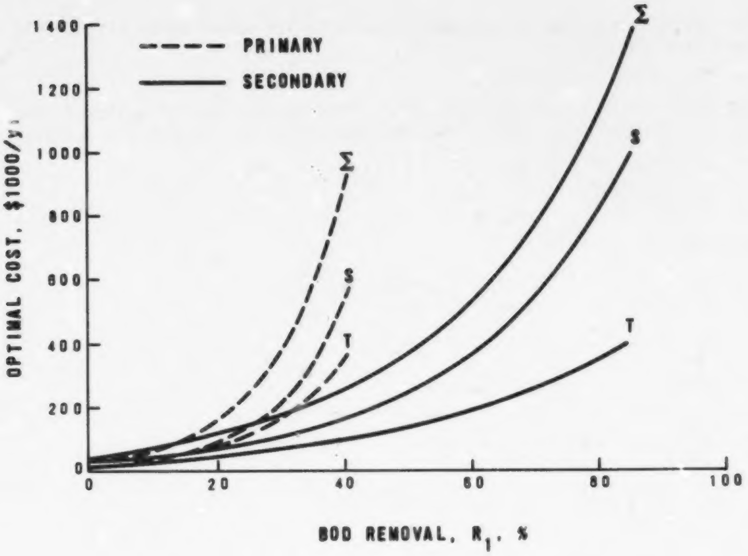


Figure 4-4. Variation of optimal costs with degree of BOD removal.

5. Compare the cost effectiveness of tertiary dry-weather treatment and secondary wet-weather treatment as alternative means of reducing BOD release from the basin.

   a. Dry-weather sewage treatment plant size

   $$D = qN_p = 7.29 \text{ Mgal/d}$$

   where q = 100 gal/capita·d (Assumption 4)
   N_p = total population = 72 947 (Table 9)

   b. Total annual incremental cost of a tertiary dry-weather treatment plant for additional organic pollution control

   $$C_{tert} = 87\ 000(D)^{0.787} \quad [3, \text{p } 64]$$
   $$= 415\ 418 \text{ \$/yr}$$

   c. Unit annual incremental cost of BOD removal from dry-weather flow by tertiary treatment,

   $$c_{tert} = \frac{C_{tert}}{(\eta_{tert} - \eta_{sec})M_{DW}(A)} \quad [3, \text{p } 66]$$
   $$= 0.65 \text{ \$/lb BOD·yr}$$

   where $(\eta_{tert} - \eta_{sec})$ = 0.10 (Assumption 5)
   M_{DW} = 704 lb BOD/acre·yr (Table 4-2)
   A = 9070 acres (Table 4-2)

   d. Area-weighted average wet-weather BOD load, excluding undeveloped areas,

   $$\bar{M}_d = \frac{10.0(3431) + 26.2(1333) + 9.9(154) + 0.1(1038)}{3431 + 1333 + 154 + 1038} \quad \text{(Table 4-2)}$$
   $$= 11.90 \text{ lb BOD/acre·yr}$$

   Note: All the areas generating this load are served by separate storm sewers in this example (Table 9, Item 10). If they had instead been served by combined sewers, the wet-weather BOD load would have been 4.12 times as large [3, p 16-17]

e. The wet-weather cost curve constants, k and β, are obtained by fitting the optimal cost function

$$Z^* = ke^{\beta R_1} \quad [3, \text{p } 50]$$

to the total cost curves of Figure 4-4. This is most easily achieved from semilogarithmic plots as in Figure 4-5. Approximating the curves in Figure 4-5 by straight lines, we obtain

$$k = Z^* \text{ at } R_1 = 0\text{, and}$$

$$\beta = \frac{\ell n(Z_a^*/Z_b^*)}{R_{1a} - R_{1b}}$$

from which

	Primary	Secondary
$k = Z_0^*$	3.24	5.93
Z_2^*	42.23	18.65
Z_2^*/Z_0^*	13.03	3.15
R_{10}	0	0
R_{12}	30	30
$\beta = \frac{\ell n(Z_2^*/Z_0^*)}{R_{12}}$	0.0856	0.0382

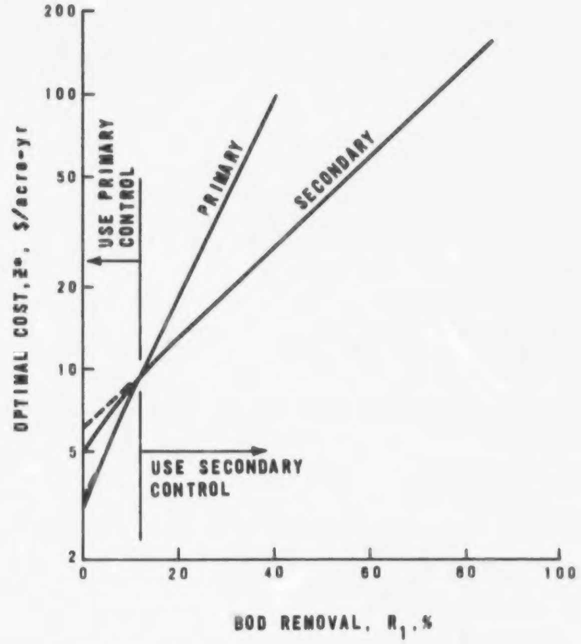


Figure 4-5. Semilogarithmic plots of the total cost data of Figure 4-4.

f. Optimum level of secondary wet-weather BOD removal to initiate before using tertiary dry-weather treatment,

$$R_1^* = \max\left[\frac{1}{\beta}\,\ell n\left\{\frac{c_{tert}\ (\bar{M}_d)}{100\ \beta(k)}\right\},\ 0\right] \qquad [3, \text{p } 67]$$

$$= \max\,[-28.1,\ 0]$$

$$= 0\%$$

Note: If this basin had been served by combined sewers instead of storm sewers, so increasing $\bar{M}_d$ to 49.03 lb BOD/acre·yr, the value of R_1^* would have been 8.9%.

6. Determine the reduction in annual costs of managing wet-weather flows made possible by integrating wet-weather treatment with dry-weather treatment and wet-weather storage, in the manners indicated in Figure 4-6, and based on Assumption 6. The cost savings in each case are achieved by a reduction in the wet-weather treatment capacity needed.

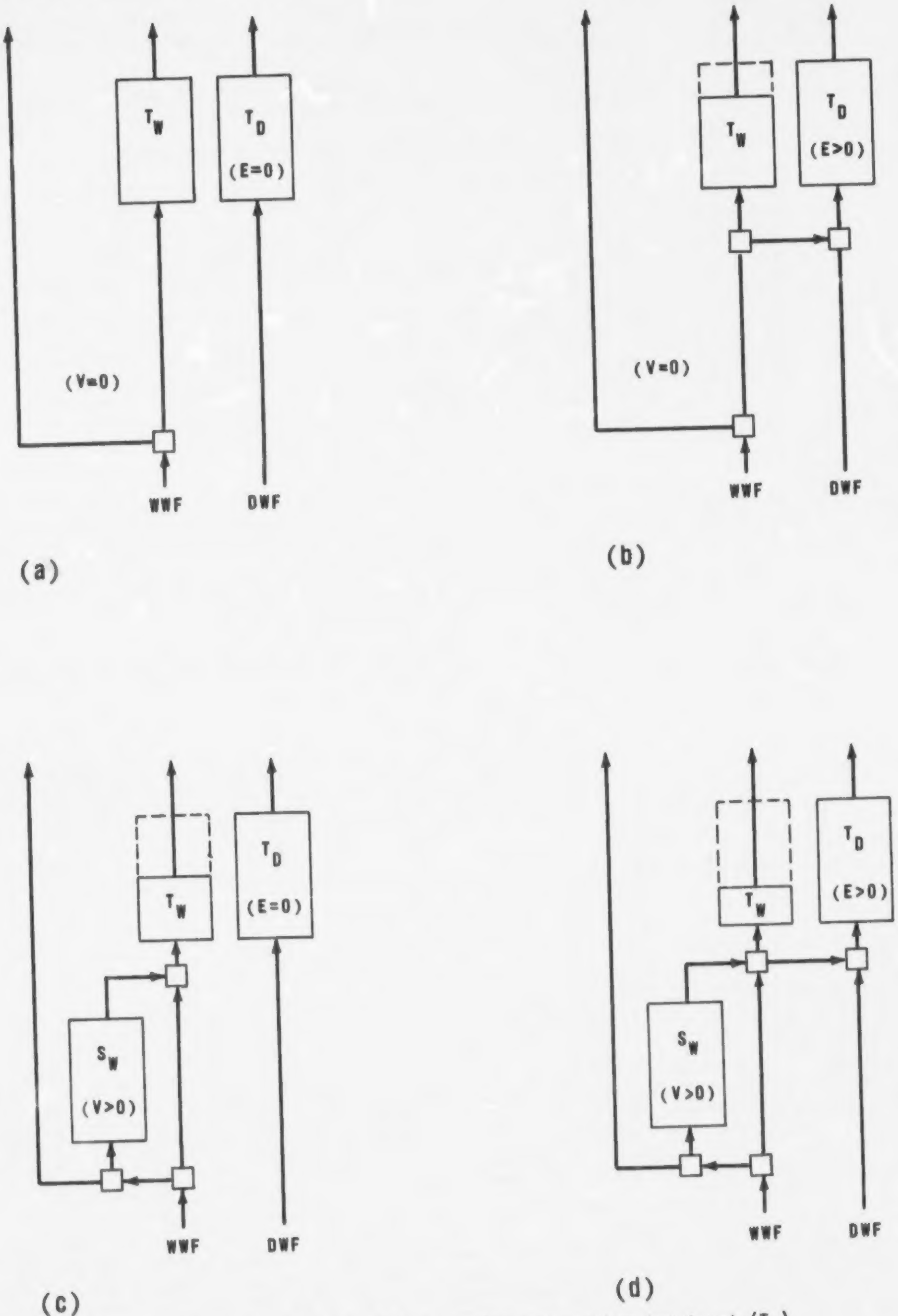

T_W
T_D
(E=0)
(V=0)
WWF
DWF
(a)
T_W
T_D
(E>0)
(V=0)
WWF
DWF
(b)
T_W
T_D
(E=0)
S_W
(V>0)
WWF
DWF
(c)
T_W
T_D
(E>0)
S_W
(V>0)
WWF
DWF
(d)

Figure 4-6. Methods of integrating wet-weather treatment (T_W) with dry-weather treatment (T_D) and wet-weather storage (S_W).

a. The treatment capacity required for treatment alone (Figure 4-6a) is given by the equation of the isoquant curve

$$T = T_1 + (T_2 - T_1)e^{-KS} \quad [3, p\ 38]$$

with storage S set to zero, namely $T = T_2$. Values of T_2 were previously computed in Step 4. The cost of this wet-weather treatment capacity is obtained by multiplying T_2 by the unit cost, \$15,000/Mgal·d (Step 3).

See Table 4-3 for results. The values in parentheses after the costs in Table 4-3 are the ratios of the costs to the respective case (a) costs.

TABLE 4-3. SUMMARY OF COSTS OF ALTERNATIVE INTEGRATION METHODS

Pollutant removal, R_1, %	10	25	50	75
Secondary treatment efficiency, η	0.85	0.85	0.85	0.85
Runoff control, R, %	11.8	29.4	58.8	88.2
Cost of treatment alone (case a), $\$10^6$/yr	0.22 (1.00)[a]	0.45 (1.00)	1.37 (1.00)	4.25 (1.00)
Cost of integrated wet- and dry-weather treatment (case b), $\$10^6$/yr	0.11 (0.50)[a]	0.34 (0.76)	1.26 (0.92)	4.13 (0.97)
Cost of integrated wet-weather storage and treatment (case c), $\$10^6$/yr	0.075 (0.34)	0.145 (0.32)	0.370 (0.27)	0.945 (0.22)
Cost of fully integrated storage and treatment (case d), $\$10^6$/yr	0.050 (0.23)	0.090 (0.20)	0.261 (0.19)	0.836 (0.20)

[a] Values in parentheses are the ratios of the costs to the respective case (a) costs.

b. When the wet- and dry-weather treatment facilities are integrated (Figure 4-6b), the flow to be processed by wet-weather treatment is reduced by one dry-weather flow (Assumption 6), or 7.29 Mgal/d (Step 5a). Therefore, the cost of the reduced wet-weather treatment facility is the case (a) cost less

7.29 (Mgal/d) 15,000 (\$/Mgal·d) = \$109 350

See results in Table 4-3.

c. The costs of optimized, integrated wet-weather treatment and storage are obtained directly from Figure 4-3.

See results in Table 4-3.

d. The costs of wet-weather treatment integrated with both wet-weather storage and dry-weather treatment are the same as for case (c), but reduced by the value of the treatment provided in the dry-weather plant. This reduction will be \$109 350 (Step 6b) or the cost of secondary treatment alone (Figure 4-4), whichever is the lesser.

See Table 4-3 for results.

The alternative costs of Table 4-3 are compared graphically in Figure 4-7.

Comments

1. A further University of Florida assumption, on the amount of on-site stormwater storage capacity provided, has not been made here. When this capacity must be limited below the requirements of Figure 4-3, optimal costs for cases (c) and (d) of Step 6 will rise.
2. The cost-effectiveness trade-off procedure of Step 5 compares the cost of extending secondary dry-weather treatment to tertiary with the incremental cost per pound BOD removed (\$0.65 in this case) at wet-weather removal efficiency R_1, rather than with the overall cost per pound (\$1.91). The former comparison overlooks the significant cost (with this method) of providing a negligible BOD removal capability (Figure 4-4). The potential user is advised to use the Step 5 procedure only with the fullest understanding of the principles involved.
3. In Step 6, the significantly lower costs of integration alternatives (c) and (d) are benefitting from a very low unit annual cost for storage (0.016 \$/gal; Step 3). This cost could equally well be one to two orders of magnitude higher. With a tenfold increase in unit storage cost, for 75% runoff control and secondary wet-weather treatment for example, the optimal storage capacity is reduced by 81% (to 4.78 Mgal); the optimal treatment capacity is increased by 670% (to 101.7 Mgal/d); and the optimal total annual cost is increased by 272% (to \$2 296 000). This is then practically as high as the case (b) annual cost (Figure 4-7).

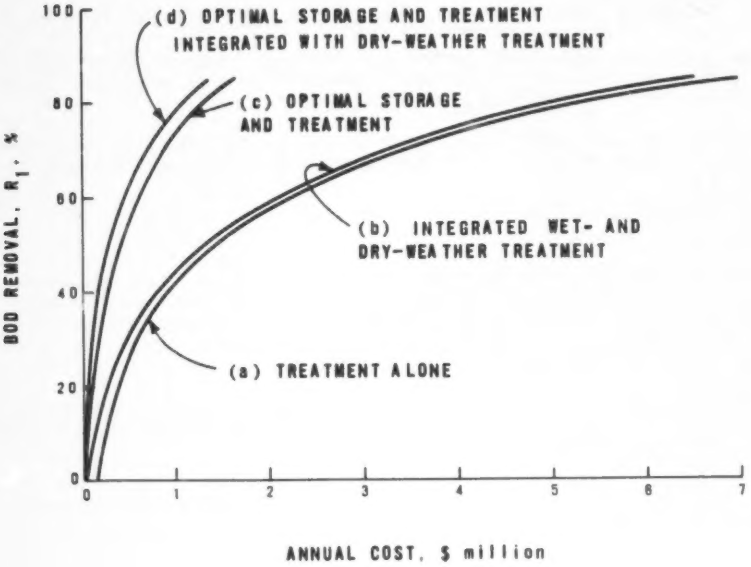


Figure 4-7. Variation of costs of various integration methods with BOD removal.

EXAMPLE PROBLEM 4-4: SELECTION OF A DESIGN STORM FROM THE HISTORICAL RECORD

Select design storms for the basin, using (a) the Simplified SWMM rainfall characterization procedure [9, Section VI] and (b) a modification of the Boston (EMMA [35]) synthetic hyetograph development procedure.

Specified Conditions

1. Period of rainfall record: 1/2/1907-9/30/1976 (69.75 years).

2. Rainfall amounts: hourly rainfall increments, recorded to the nearest 0.01 in., at a base rain gage within or adjacent to the study area.

Assumptions

1. An independent storm event is identified by preceding and following dry periods of at least 6 hours duration.

2. For the modified Boston procedure (b), a synthetic hyetograph with a 1 year return period is preferred for the reasons given in [35, Appendix B].

Solution

1. Acquire the appropriate hourly precipitation tape file from the National Oceanic and Atmospheric Administration (previously the U.S. Weather Bureau), Environmental Data Service, in Asheville, N.C. Print the entire historical record.

2. Using the computer program EVENT [9, Section VI], define the independent storm events using Assumption 1. Using program LISTSQ [9, Section VI], tabulate these storms with event characteristics such as: date, starting hour, duration, total precipitation, maximum hourly rainfall and the hour in which it occurred, and elapsed days since the previous storm.

3. Using an IBM package sorting program, SORT [9, Section VI], rank the N storms with the largest total precipitation, where N = 1.5 x number of years of rainfall record. In this case N = 1.5(69.75) = 105 storms. Use program LISTRK [9, Section VI], to list the ranked files.

4. Determine the average intensity (total precipitation/storm duration) for each of the N storms in Step 3. Apply programs SORT and LISTRK a second time, to rank and list the storms by average intensity. Select for further consideration the upper 50% of this ranking, containing the larger average intensities. These will be the storms that generally place the greatest stress on the storage/treatment facilities.

5. Apply programs SORT and LISTRK a third time to rank and list the remaining storms by duration. Results are presented in Table 4-4. Identify the mean and median durations; these, from Table 4-4, are:

Mean duration:	30.34 h
Median duration:	30.00 h

6. Select from the Step 5 ranking about 10 storms with durations in the neighborhood of the mean and/or median, and, if possible, having similar shape characteristics (time to peak, time distribution of rain). The 17 storms selected for the study area are indicated by shading in Table 4-4 (duration ranks = 17 to 30). Tabulate, from Step 1, the hourly rainfall for each; results are given in Table 4-5.

7. To complete the Simplified SWMM rainfall characterization procedure (a), select from the storms of Step 6 that one (those) which is (are) judged to be most representative of them all, to be the design storm(s).

   Due to the variety of rainfall distributions at the study area (Table 4-5), two storms, labeled S1 and S2 in Tables 4-4 and 4-5, were selected as being representative. Their hyetographs are plotted in Figure 4-8.

TABLE 4-4. STEP 5 RESULTS OF STORM EVENT RANKING

Year	Month	Day	Start hour	Duration: Hours	Duration: Rank	Total rainfall: Inches	Total rainfall: Rank	Average intensity, in./h	Maximum hour rainfall: Amount, in.	Maximum hour rainfall: Hour after start	Days since last storm	Selection
1945	12	26	24	56	1	3.76	7	0.067	0.36	11	0	
1956	1	13	15	50	2	4.70	2	0.094	0.48	24	3	
1908	2	1	1	43	3	3.04	20	0.071	0.15	1	3	
1967	1	20	5	42	4	4.55	3	0.108	0.41	39	9	
1914	2	19	14	39	5	2.98	23	0.076	0.32	13	0	
1916	1	2	8	39	5	3.67	10	0.094	0.40	16	1	
1943	1	19	24	39	5	3.02	22	0.077	0.35	35	22	
1942	11	16	12	38	8	3.10	18	0.082	0.27	10	1	
1958	4	1	18	38	8	2.64	37	0.069	0.95	20	0	
1916	1	12	20	37	10	2.53	42	0.068	0.51	32	2	
1973	1	8	4	37	10	3.25	16	0.088	0.45	13	12	
1950	1	16	4	36	12	2.81	29	0.078	0.31	18	2	
1918	11	17	18	35	13	2.32	55	0.066	0.45	17	2	
1924	12	20	18	35	13	3.58	12	0.102	0.56	20	4	
1950	12	2	12	34	15	2.58	40	0.076	0.26	14	2	
1921	12	25	21	33	16	2.26	57	0.068	0.29	27	0	
1908	1	19	24	32	17	2.18	63	0.068	0.26	19	0	
1911	1	13	9	32	17	2.60	39	0.081	0.46	9	0	
1919	2	9	24	32	17	3.72	8	0.116	0.53	11	0	
1926	4	4	3	32	17	2.70	34	0.084	0.39	7	17	
1940	2	26	8	32	17	2.73	31	0.085	0.24	30	0	
1945	1	31	19	32	17	2.70	35	0.084	0.25	11	0	
1969	10	14	16	32	17	2.36	53	0.074	0.47	20	0	S1
1926	1	30	13	31	24	2.49	46	0.080	0.21	13	2	
1958	2	18	3	31	24	2.15	69	0.069	0.29	17	3	
1931	12	26	12	30	26	2.81	29	0.094	0.33	21	1	
1962	10	12	17	30	26	3.34	15	0.111	0.63	2	0	
1936	2	10	16	29	28	3.19	17	0.110	0.47	18	9	S2
1940	3	29	15	29	28	3.86	5	0.133	0.28	3	0	
1907	3	18	4	28	30	1.92	103	0.069	0.20	10	1	
1937	1	19	10	28	30	2.04	87	0.073	0.24	10	5	
1941	12	27	20	28	30	2.06	84	0.074	0.26	21	0	
1952	12	19	1	28	30	2.10	76	0.075	0.41	21	0	
1915	3	27	10	27	34	1.85	99	0.072	0.27	2	0	
1921	1	16	12	27	34	2.70	33	0.100	0.39	18	4	
1958	3	22	19	27	34	2.53	43	0.094	0.39	9	0	
1937	3	20	23	26	37	1.94	100	0.075	0.30	8	0	
1969	12	19	6	26	37	3.03	21	0.117	0.73	23	0	
1923	4	1	22	25	39	2.06	82	0.082	0.33	18	1	
1972	11	13	4	25	39	2.48	47	0.099	0.51	21	3	
1912	3	4	23	24	41	2.09	78	0.087	1.05	9	0	
1962	2	14	8	24	41	1.92	105	0.080	0.48	4	1	
1970	1	20	14	24	41	2.31	56	0.096	0.25	4	0	
1953	4	26	14	23	44	2.36	52	0.103	0.40	14	7	
1973	11	5	11	23	44	2.72	32	0.118	0.35	5	14	
1941	3	30	8	22	46	2.22	61	0.101	0.40	10	0	
1915	12	2	23	21	47	3.40	14	0.162	0.38	12	0	
1920	12	23	6	21	47	2.02	89	0.096	0.30	19	2	
1959	9	17	21	21	47	2.06	85	0.098	0.43	13	24	
1913	11	18	2	20	50	2.21	62	0.110	0.33	9	5	
1922	11	6	7	20	50	2.09	79	0.104	0.35	17	6	
1926	1	28	5	19	52	2.38	48	0.125	0.37	19	11	
1952	12	6	17	16	53	1.92	104	0.120	0.38	8	1	

TABLE 4-5. HOURLY RAINFALL (IN HUNDREDTHS OF AN INCH) FOR STORMS SELECTED IN STEP 6

Storm hour	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	Selection
Date																																	
01/19/1908	1	6	8	11	7	9	7	1	1	0	1	1	0	5	3	10	16	18	26	22	22	12	11	7	5	1	1	0	1	0	1	4	
01/13/1911	2	5	8	10	9	7	7	22	46	19	7	7	6	11	4	3	6	7	5	10	10	6	7	19	6	3	1	1	1	2	1	2	
02/09/1919	1	0	1	11	17	11	2	5	13	17	53	32	24	9	13	16	20	7	3	0	1	11	3	7	10	11	35	11	4	12	2	10	
04/04/1926	7	4	3	5	10	13	39	22	15	6	0	0	1	1	0	0	1	0	1	22	31	8	13	4	10	16	1	4	24	5	3	1	
02/26/1940	3	1	4	4	2	6	10	9	9	16	7	9	11	20	17	13	16	18	8	1	2	2	22	1	0	4	1	4	8	24	11	10	
01/31/1945	4	2	0	1	12	18	2	16	23	16	25	12	4	8	16	9	5	7	0	0	1	19	6	10	13	4	1	0	11	7	17	1	
10/14/1969	1	0	0	1	0	0	0	1	2	3	2	5	5	4	8	7	27	24	35	47	5	1	2	16	14	25	0	0	0	1	0	1	S1
01/30/1926	4	15	14	13	20	16	4	2	0	1	4	5	21	19	21	20	14	6	1	11	9	0	1	10	0	0	1	0	16	0	1		
02/18/1958	2	3	6	12	5	5	10	6	11	1	2	20	1	7	12	2	29	21	7	11	2	1	1	4	1	2	3	4	22	1	1		
12/26/1931	2	2	5	2	0	1	5	4	10	13	17	17	9	9	8	28	21	13	21	19	33	11	2	2	6	5	2	8	5	1			
10/12/1962	10	63	21	20	11	15	9	5	10	9	10	10	14	9	16	9	4	10	7	7	5	14	5	8	0	1	1	17	3	1			
02/10/1936	2	11	30	12	12	26	23	3	30	13	4	1	21	6	2	47	18	26	9	2	0	0	0	0	11	1	0	3	5				S2
03/29/1940	2	27	18	19	22	17	9	20	12	12	6	12	23	16	23	14	16	15	19	12	16	9	8	3	6	11	2	6	1				
03/18/1907	13	10	3	8	9	7	6	9	6	20	13	4	2	0	4	6	2	3	13	7	10	10	10	5	8	3	0	1					
01/10/1937	3	7	9	10	11	10	9	15	17	24	8	2	0	1	1	15	6	7	5	7	1	2	8	6	8	6	3	3					
12/27/1941	8	0	2	0	0	5	2	0	6	24	0	1	0	0	0	23	6	0	8	17	26	25	7	3	17	13	10	3					
12/19/1952	3	2	1	4	0	4	5	7	11	6	2	5	3	25	30	26	3	0	0	1	41	19	0	0	0	0	1	11					
No. (P>20)	0	2	2	0	1	1	2	2	3	2	2	1	4	1	3	4	3	3	3	3	5	1	1	0	0	1	1	0	2	1	0	0	

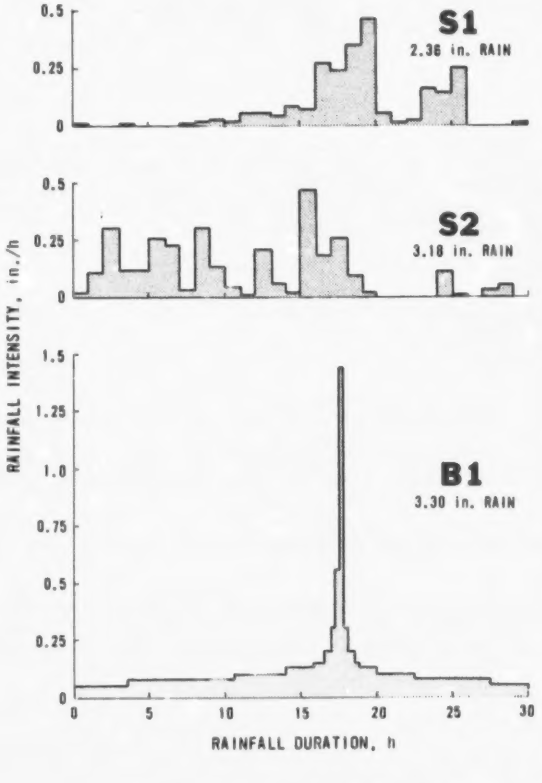


Figure 4-8. Design storm hyetographs.

8. To complete the modified Boston synthetic hyetograph procedure [35, Appendix A]:

   a. Locate the point in time where the maximum rainfall intensity will occur. Because of the great variety of rainfall distributions at the study area (Table 4-5), the average distribution method [35, Appendix A, Step 4] did not give definitive results. Instead, the maximum intensity was located by inspecting the distribution of the top 10% of the hourly rainfall intensities, in this case those greater than 0.20 in./h. From Table 4-5, bottom line, select hour 18 (the middle of a cluster).

   b. Determine from the U.S. Weather Bureau rainfall frequency atlas [36] the rainfall depths at the basin location for various durations and for a 1 year return period (Assumption 2), by interpolating between isopluvial lines. Compute the 5, 10, and 15 minute duration rainfalls from the 30 minute duration rainfall using the appropriate ratios [36, Table 3]. Rank the distribution of rainfalls by intervals, as follows:

Duration	Precipitation, in.	Interval	Remaining precipitation, in.	Interval intensity, in./h
5 min	0.185	First 5 min	0.185	2.22
10 min	0.285	Next 5 min	0.100	1.20
15 min	0.36	Next 5 min	0.075	0.90
30 min	0.50	Next 15 min	0.14	0.56
1 h	0.65	Next 30 min	0.15	0.30
2 h	0.85	Next 1 h	0.20	0.20
3 h	1.0	Next 1 h	0.15	0.15
6 h	1.4	Next 3 h	0.4	0.13
12 h	2.0	Next 6 h	0.6	0.10
24 h	3.0	Next 12 h	1.0	0.08
>24 h	--	Next 6 h	--	~0.05

   Note: The interval intensities must steadily decrease. To achieve this it may be necessary to slightly adjust the interpolated precipitation data.

   c. Select a time-step size, Δt. For subsequent simulation with the SWMM model (Example Problem 4-8), choose Δt ≈ duration/100. Therefore

$$\Delta t \approx \frac{30\text{ h}}{100} = 18\text{ min}$$

$$\Delta t = 15\text{ min, say}$$

   d. Distribute the precipitation from Step 8b, by time-step (Step 8c), throughout the storm duration in proportion to the total time preceding and following the peak. Start with the time-step containing the hyetograph peak. Convert the ordinates of the resulting graph to rainfall intensity, to yield the design hyetograph. The resulting design hyetograph is labeled B1 in Figure 4-8.

Comments

1. The total precipitation in each of the three design storms is as follows:

S1	2.36 in.
S2	3.18 in.
B1	3.30 in.

   Except for the short, intense peak, the synthetic storm B1 is more uniform than the others.

2. The great variety of totally different storm patterns which occur along the West Coast (Table 4-5) requires that a number of different design storm patterns be used for model calibration there.

3. Rainfall analysis methods obviously must depend on the nature of the data, and hence the study area location.

4. A good alternative to the above design storm approach is described in an EPA report [2] released after the completion of this study. It is based on a continuous characterization in terms of the percentage of the total annual precipitation volume and/or storms which could be treated.

EXAMPLE PROBLEM 4-5: INITIAL ASSESSMENT OF THE MANAGEMENT PROBLEM (METCALF & EDDY CONTINUOUS SIMULATION METHOD, SIMPLIFIED SWMM MODEL [9])

Determine the monthly average wet-weather flows and pollutant loads from the basin for two selected years for (a) separate storm sewers and (b) assuming the area is served bv combined sewers.

Specified Conditions

1. Rainfall years: 1969-1970, 1970-1971
2. Rainfall amounts: Daily rainfall increments at San Jose City Hall, increased by 40% (direct extrapolation by isohyetal lines)
3. Average annual runoff quantity and quality characteristics, by land use:
    a. Separate sewers:

LAND USE	AREA ACRES	K-FACTOR	POLLUTANT CONCENTRATION IN SURFACE RUNOFF, MG/L					
			BOD	SS	VSS	TOT N	TOT P	OTHERS
RESIDENTIAL	3431.	0.27	10.40	211.50	123.00	1.70	0.44	0.0
RESIDENTIAL MULTI	0.	0.0	0.0	0.0	0.0	0.0	0.0	0.0
COMMERCIAL	1333.	0.50	41.50	288.10	181.70	3.84	0.98	0.0
INDUSTRIAL	154.	0.60	15.80	377.60	186.80	3.58	0.91	0.0
OPEN	4127.	0.23	0.20	5.00	4.80	0.11	0.02	0.0
AGRICULTURAL	0.	0.0	0.0	0.0	0.0	0.0	0.0	0.0

    b. Combined sewers:

LAND USE	AREA ACRES	K-FACTOR	POLLUTANT CONCENTRATION IN SURFACE RUNOFF, MG/L					
			BOD	SS	VSS	TOT N	TOT P	OTHERS
RESIDENTIAL	3431.	0.27	42.70	871.90	505.70	7.01	1.80	0.0
RESIDENTIAL MULTI	0.	0.0	0.0	0.0	0.0	0.0	0.0	0.0
COMMERCIAL	1333.	0.50	171.30	1191.20	751.30	15.83	4.05	0.0
INDUSTRIAL	154.	0.60	64.90	1557.00	770.80	14.82	3.78	0.0
OPEN	4127.	0.23	0.90	20.40	19.90	0.46	0.08	0.0
AGRICULTURAL	0.	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Note 1: The different runoff coefficients (K-factors) represent variations in perviousness with land use; the values used here had been previously calibrated by adjusting quantity and quality predictions for selected storms.

Note 2: The pollutant concentrations [9, Appendix D] are derived from the same source as those of Example Problem 4-1. The population density function, f , to adjust pollution concentrations from residential areas, was found to be 1.0, based on the basin's developed population density of 12.25 people/acre (72 947 people/[9045-3114] acres).

Assumptions

1. The average annual runoff is a prescribed fraction of the annual precipitation; this fraction varies only with type of land use.
2. Pollutant loads are determined by prescribed runoff concentrations [9, Table D-2].

Solution

1. Prepare input for the Simplified SWMM computer program, from the data listed under Specified Conditions.
2. Execute the program (0.01 min CPU time, $0.45 total cost, on IBM 370/168).
3. Computer output is presented in Table 4-6.

TABLE 4-6. COMPUTER OUTPUT FOR EXAMPLE PROBLEM 4-5

a. Separate sewers

```
  YEAR 1969-1970                                                      MONTHLY SUMMARY
_________________________________________________________________________________________________________________________________

                                                                                        POLLUTANT LOAD, THOUSANDS OF POUNDS
MONTH DAY  RAIN  RAIN  RUNOFF   OVERFLOW  TREATED  OVERFL  TREAT  MAX STORAGE  ____________________________________________________
            IN.  DAYS   Mgal      Mgal      Mgal    DAYS   DAYS      Mgal         BOD      SS      VSS    TOT N   TOT P   OTHERS
_____ ___  ____  ____  ______   ________  _______  ______  _____  ___________  _______ _______ _______ _______ _______ _______

   7   31  0.0     0      0.0       0.0      0.0     0      0.0         0.0        0.      0.      0.      0.      0.      0.
   8   31  0.0     0      0.0       0.0      0.0     0      0.0         0.0        0.      0.      0.      0.      0.      0.
   9   30  0.20    1     14.0      14.0      0.0     1      0.0         0.0        2.     19.     11.      0.      0.      0.
  10   31  0.87    5     62.1      62.1      0.0     5      0.0         0.0        8.     84.     51.      1.      0.      0.
  11   30  1.30    3     93.1      93.1      0.0     3      0.0         0.0       11.    126.     76.      1.      0.      0.
  12   31  2.20   10    157.2     157.2      0.0    10      0.0         0.0       19.    213.    128.      2.      1.      0.
   1   31  5.63   15    402.6     402.6      0.0    15      0.0         0.0       50.    545.    328.      6.      1.      0.
   2   28  2.14    9    153.2     153.2      0.0     9      0.0         0.0       19.    208.    125.      2.      1.      0.
   3   31  3.05    5    218.3     218.3      0.0     5      0.0         0.0       27.    296.    178.      3.      1.      0.
   4   30  0.29    3     21.0      21.0      0.0     3      0.0         0.0        3.     28.     17.      0.      0.      0.
   5   31  0.03    1      2.0       2.0      0.0     1      0.0         0.0        0.      3.      2.      0.      0.      0.
   6   30  0.14    1     10.0      10.0      0.0     1      0.0         0.0        1.     14.      8.      0.      0.      0.


YEARLY     15.85  53   1133.8    1133.8      0.0    53      0.0         0.0      140.   1536.    922.     16.      4.      0.
```

```
  YEAR 1970-1971                                                      MONTHLY SUMMARY
_________________________________________________________________________________________________________________________________

                                                                                        POLLUTANT LOAD, THOUSANDS OF POUNDS
MONTH DAY  RAIN  RAIN  RUNOFF   OVERFLOW  TREATED  OVERFL  TREAT  MAX STORAGE  ____________________________________________________
            IN.  DAYS   Mgal      Mgal      Mgal    DAYS   DAYS      Mgal         BOD      SS      VSS    TOT N   TOT P   OTHERS
_____ ___  ____  ____  ______   ________  _______  ______  _____  ___________  _______ _______ _______ _______ _______ _______

   7   31  0.0     0      0.0       0.0      0.0     0      0.0         0.0        0.      0.      0.      0.      0.      0.
   8   31  0.0     0      0.0       0.0      0.0     0      0.0         0.0        0.      0.      0.      0.      0.      0.
   9   30  0.0     0      0.0       0.0      0.0     0      0.0         0.0        0.      0.      0.      0.      0.      0.
  10   31  0.49    3     35.1      35.1      0.0     3      0.0         0.0        4.     47.     29.      1.      0.      0.
  11   30  8.71   12    623.0     623.0      0.0    12      0.0         0.0       77.    844.    507.      9.      2.      0.
  12   31  5.45   15    389.6     389.6      0.0    15      0.0         0.0       48.    528.    317.      6.      1.      0.
   1   31  1.27    8     91.1      91.1      0.0     8      0.0         0.0       11.    123.     74.      1.      0.      0.
   2   28  0.76    4     54.1      54.1      0.0     4      0.0         0.0        7.     73.     44.      1.      0.      0.
   3   31  2.18    6    156.2     156.2      0.0     6      0.0         0.0       19.    212.    127.      2.      1.      0.
   4   30  0.0     0      0.0       0.0      0.0     0      0.0         0.0        0.      0.      0.      0.      0.      0.
   5   31  0.11    5      8.0       8.0      0.0     5      0.0         0.0        1.     11.      7.      0.      0.      0.
   6   30  0.0     0      0.0       0.0      0.0     0      0.0         0.0        0.      0.      0.      0.      0.      0.


YEARLY     18.97  53   1357.1    1357.1      0.0    53      0.0         0.0      167.   1838.   1104.     20.      5.      0.
```

b. Combined sewers

```
  YEAR 1969-1970                                                      MONTHLY SUMMARY
_________________________________________________________________________________________________________________________________

                                                                                        POLLUTANT LOAD, THOUSANDS OF POUNDS
MONTH DAY  RAIN  RAIN  RUNOFF   OVERFLOW  TREATED  OVERFL  TREAT  MAX STORAGE  ____________________________________________________
            IN.  DAYS   Mgal      Mgal      Mgal    DAYS   DAYS      Mgal         BOD      SS      VSS    TOT N   TOT P   OTHERS
_____ ___  ____  ____  ______   ________  _______  ______  _____  ___________  _______ _______ _______ _______ _______ _______

   7   31  0.0     0      0.0       0.0      0.0     0      0.0         0.0        0.      0.      0.      0.      0.      0.
   8   31  0.0     0      0.0       0.0      0.0     0      0.0         0.0        0.      0.      0.      0.      0.      0.
   9   30  0.20    1     14.0      14.0      0.0     1      0.0         0.0        7.     78.     47.      1.      0.      0.
  10   31  0.87    5     62.1      62.1      0.0     5      0.0         0.0       32.    347.    208.      4.      1.      0.
  11   30  1.30    3     93.1      93.1      0.0     3      0.0         0.0       47.    521.    313.      5.      1.      0.
  12   31  2.20   10    157.2     157.2      0.0    10      0.0         0.0       80.    879.    528.      9.      2.      0.
   1   31  5.63   15    402.6     402.6      0.0    15      0.0         0.0      205.   2251.   1351.     23.      6.      0.
   2   28  2.14    9    153.2     153.2      0.0     9      0.0         0.0       78.    857.    514.      9.      2.      0.
   3   31  3.05    5    218.3     218.3      0.0     5      0.0         0.0      111.   1221.    733.     12.      3.      0.
   4   30  0.29    3     21.0      21.0      0.0     3      0.0         0.0       11.    118.     71.      1.      0.      0.
   5   31  0.03    1      2.0       2.0      0.0     1      0.0         0.0        1.     11.      7.      0.      0.      0.
   6   30  0.14    1     10.0      10.0      0.0     1      0.0         0.0        5.     56.     34.      1.      0.      0.


YEARLY     15.85  53   1133.8    1133.8      0.0    53      0.0         0.0      577.   6338.   3804.     64.     17.      0.
```

Table 4-6. (Concluded)

YEAR 1970-1971							MONTHLY SUMMARY									
										POLLUTANT LOAD, THOUSANDS OF POUNDS					
MONTH	DAY	RAIN IN.	RAIN DAYS	RUNOFF Mgal	OVERFLOW Mgal	TREATED Mgal	OVERFL DAYS	TREAT DAYS	MAX STORAGE Mgal	BOD	SS	VSS	TOT N	TOT P	OTHERS
7	31	0.0	0	0.0	0.0	0.0	0	0.0	0.0	0.	0.	0.	0.	0.	0.
8	31	0.0	0	0.0	0.0	0.0	0	0.0	0.0	0.	0.	0.	0.	0.	0.
9	30	0.0	0	0.0	0.0	0.0	0	0.0	0.0	0.	0.	0.	0.	0.	0.
10	31	0.49	3	35.1	35.1	0.0	3	0.0	0.0	18.	196.	118.	2.	1.	0.
11	30	8.71	12	623.0	623.0	0.0	12	0.0	0.0	317.	3483.	2090.	35.	9.	0.
12	31	5.45	15	389.6	389.6	0.0	15	0.0	0.0	198.	2178.	1307.	22.	6.	0.
1	31	1.27	8	91.1	91.1	0.0	8	0.0	0.0	46.	510.	306.	5.	1.	0.
2	28	0.76	4	54.1	54.1	0.0	4	0.0	0.0	28.	302.	181.	3.	1.	0.
3	31	2.18	6	156.2	156.2	0.0	6	0.0	0.0	79.	873.	524.	9.	2.	0.
4	30	0.0	0	0.0	0.0	0.0	0	0.0	0.0	0.	0.	0.	0.	0.	0.
5	31	0.11	5	8.0	8.0	0.0	5	0.0	0.0	4.	45.	27.	0.	0.	0.
6	30	0.0	0	0.0	0.0	0.0	0	0.0	0.0	0.	0.	0.	0.	0.	0.
YEARLY		18.97	53	1357.1	1357.1	0.0	53	0.0	0.0	690.	7587.	4554.	77.	21.	0.

Comments

1. The average annual basin runoff coefficient is 0.29 (2491 Mgal/34.82 in.).

2. The probable magnitude of the monthly and annual loads of significant wet-weather pollutants are listed in Table 4-6, for both (a) separate storm sewers and (b) combined sewers.

3. The annual wet-weather BOD load will be about 153 000 lb (separate sewers) or 633 000 lb (combined sewers), of which 71% originates from commercial land use areas and 25% from residential areas.

4. The average annual BOD concentration is 14.8 mg/L (separate sewers) or 61.0 mg/L (combined sewers).

5. These BOD loads and concentrations are greater than the results of Example Problem 4-1 for the following reasons:
   - This example yields a higher overall runoff coefficient
   - In particular, the runoff coefficient for commercial land use areas, which have by far the highest washoff BOD concentration, is notably larger in this example
   - Pollutant washoff is reduced in Example Problem 4-1 by 30% through the incorporation of a street sweeping effectiveness factor

EXAMPLE PROBLEM 4-6: PRELIMINARY PLANNING OF INTEGRATED STORAGE AND TREATMENT (METCALF & EDDY CONTINUOUS SIMULATION METHOD, SIMPLIFIED SWMM MODEL [9])

Determine the variation of storage capacities and costs required to yield various levels of BOD removal and overflow control with treatment capacities of (a) one-half dry-weather flow, and (b) four dry-weather flows.

Specified Conditions

1. Rainfall period: 1951-1976 (25 years)

2. Rainfall amounts: Daily rainfall increments at San Jose City Hall, increased by 40% (direct extrapolation by isohyetal lines).

3. Drainage basin served by separate sewers only.

4. Average annual runoff quantity and quality characteristics are the same as for Example Problem 4-5, Specified Condition 3a.

Assumptions

1. The average annual runoff is a prescribed fraction of the annual precipitation; this fraction varies only with type of land use.
2. Pollutant loads are determined by prescribed runoff concentrations [9, Table D-2].
3. Quantities passing through storage and treatment are determined from daily time-step computations.
4. One dry-weather flow equals 7.3 Mgal/d (Example Problem 4-3, Step 5a).
5. Representative annual cost (debt service plus operation and maintenance) of storage = \$0.01626/gal·yr (Example Problem 4-3, Step 3).
6. Stormwater treatment removes 85% of each pollutant. Resulting treated effluent concentrations (using Specified Condition 4) are:

	mg/L
BOD	2.22
SS	24.35
VSS	14.63
Total N	1.73
Total P	0.07

Solution

1. Prepare input for the Simplified SWMM computer program, from the data listed under Specified Conditions and Assumption 6, plus the following treatment and storage capacities (Assumption 4):

Treatment (withdrawal) rate, Mgal/d	Storage capacity, Mgal	Treatment (withdrawal) rate, Mgal/d	Storage capacity, Mgal
3.6	0.0	29.2	0.0
3.6	2.0	29.2	2.0
3.6	5.0	29.2	5.0
3.6	20.0	29.2	20.0
3.6	50.0	29.2	50.0

2. Execute the program (0.44 min CPU time, \$14.00 total cost, on IBM 370/168).
3. Sample computer output is presented in Table 4-7.

TABLE 4-7. SAMPLE COMPUTER OUTPUT FOR 2.0 Mgal STORAGE
AND 3.6 Mgal/d TREATMENT CAPACITY

```
   YEAR  1970 - 1971                         MONTHLY  SUMMARY

                                CALABAZAS CREEK            SEPARATE-STOR/TREAT

                                                                          POLLUTANT LOAD, THOUSANDS OF POUNDS
MONTH DAY  RAIN  RAIN  RUNOFF  OVERFLOW  TREATED  OVERFL  TREAT  MAX STORAGE  -------------------------------------------
            IN   DAYS  Mgal     Mgal      Mgal    DAYS    DAYS     Mgal        BOD    SS     VSS   TOT N  TOT P  OTHERS

  7   31  0.0     0      0.0      0.0      0.0      0      0.0       0.0         0.     0.     0.     0.     0.     0.
  8   31  0.0     0      0.0      0.0      0.0      0      0.0       0.0         0.     0.     0.     0.     0.     0.
  9   30  0.0     0      0.0      0.0      0.0      0      0.0       0.0         0.     0.     0.     0.     0.     0.
 10   31  0.49    3     35.1     20.3     14.8      2      4.1       2.0         3.    30.    18.     1.     0.     0.
 11   30  8.71   12    623.0    577.4     43.6     11     12.1       2.0        72.   791.   475.     9.     2.     0.
 12   31  5.45   15    389.6    333.4     58.2     12     16.2       2.0        42.   463.   278.     6.     1.     0.
  1   31  1.27    8     91.1     66.1     25.0      5      6.9       2.0         9.    95.    57.     1.     0.     0.
  2   28  0.76    4     54.1     34.3     19.8      3      5.5       2.0         5.    50.    30.     1.     0.     0.
  3   31  2.18    6    156.2    128.6     27.6      5      7.7       2.0        16.   180.   108.     2.     0.     0.
  4   30  0.0     0      0.0      0.0      0.0      0      0.0       0.0         0.     0.     0.     0.     0.     0.
  5   31  0.11    5      8.0      0.0      8.0      0      2.2       0.0         0.     2.     1.     0.     0.     0.
  6   30  0.0     0      0.0      0.0      0.0      0      0.0       0.0         0.     0.     0.     0.     0.     0.


YEARLY   18.97   53   1357.1   1160.1    197.0     38     54.7       2.0       147.  1611.   968.    20.     4.     0.
```

4. Calculate 25 year BOD removal efficiencies and storage costs (Assumption 5) from the average annual flows:

Storage capacity, Mgal	Treatment rate, Mgal/d	Overflow, Mgal/yr	Treated, Mgal/yr	Overflows, No.	BOD			Annual storage cost, $\$10^6$/yr
					Released, 1000 lb/yr	Removed, 1000 lb/yr	Removed, %	
0	0	1346	0	1359	166.0	0	0	0
0	3.6	1175	171	1021	148.1	17.9	10.8	0
2	3.6	1132	214	961	143.6	22.4	13.5	0.033
5	3.6	1081	265	883	138.2	27.8	16.7	0.081
20	3.6	940	406	721	123.5	42.5	25.6	0.325
50	3.6	802	544	609	109.0	57.0	34.3	0.813
0	29.2	546	800	377	82.1	83.9	50.5	0
2	29.2	524	822	355	79.9	86.1	51.9	0.033
5	29.2	495	851	331	76.8	89.2	53.7	0.081
20	29.2	370	976	263	63.7	102.3	61.6	0.325
50	29.2	217	1129	144	47.7	118.3	71.3	0.813

These results are presented graphically in Figures 4-9 and 4-10.

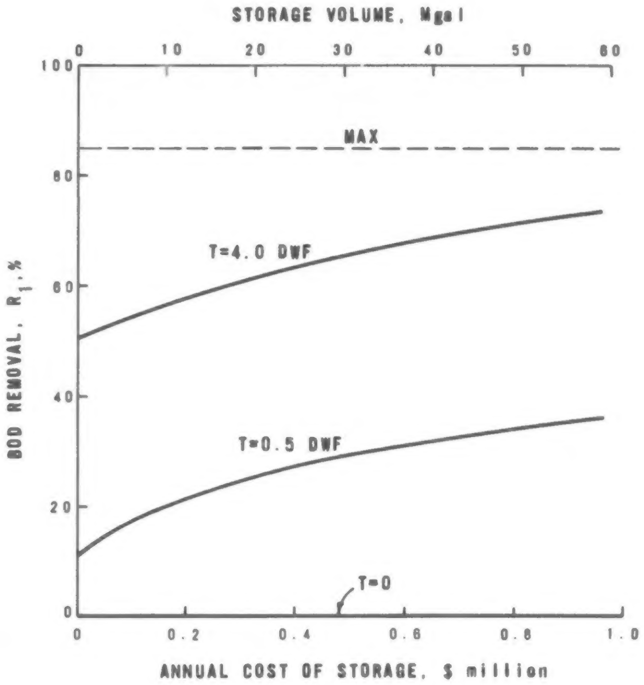


Figure 4-9. Variation of 1951-1976 BOD removal efficiencies with size and cost of storage.

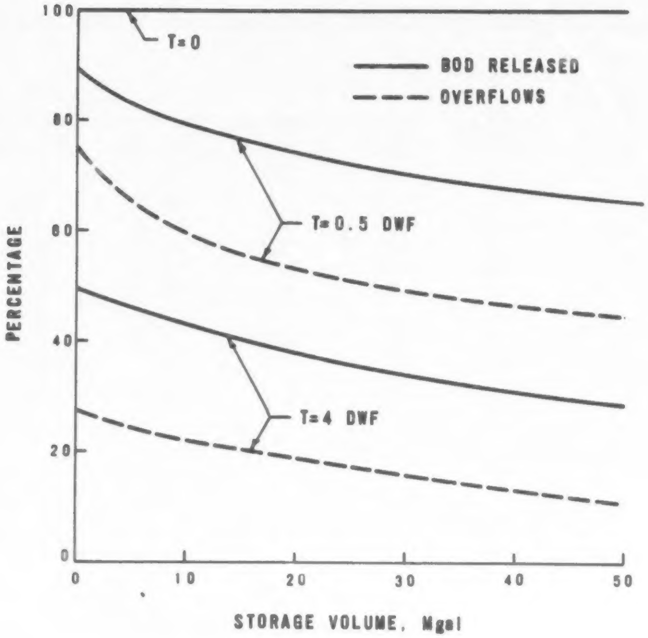


Figure 4-10. Variation of BOD releases and number of overflows during 1951-1976 with storage volume.

Comments

1. Increases in BOD removal are seen in Figure 4-9 to be relatively small for large increases in storage cost. The incremental benefits diminish with increasing storage cost.

2. The number of overflow (extreme) events that occur during the 25 year period is seen in Figure 4-10 to be reduced by storage and treatment far more significantly than is total BOD release.

3. The simplifying assumptions do not account for: (1) treatment accomplished in storage and (2) variability of treatment performance with the number and duration of treatment periods, startup effects, and mixing in storage. Consideration of these points through in-depth analysis may show improved storage benefits.

4. The inability of the Simplified SWMM model to simulate flow variations within each day (Assumption 3) suggests that overflows sufficiently short to not be modeled will somewhat reduce the BOD removals. It appears that this reduction might decrease with increasing storage volume.

EXAMPLE PROBLEM 4-7: CALIBRATE DETAILED EVENT MODEL ON THE TEST AREA (EPA SWMM MODEL [7, 28])

Adjust model parameters so that it predicts runoff quantities and qualities in good agreement with observations of two selected storms.

Specified Conditions

1. Storm dates (with outflow quantity and quality measurements):
    a. December 29-30, 1976
    b. January 2, 1977

2. Rainfall amounts and locations: 15 minute rainfall increments, recorded to the nearest 0.01 in., at a number of county rain gages surrounding the basin.

3. Drainage basin served by separate sewers only.

Solution

1. Divide the basin into subcatchments that correspond to the drainage network, so that each subarea has near-uniform land use and topography characteristics. Resulting study basin subcatchments are depicted in Figure 4-11 (see also Figure 13).

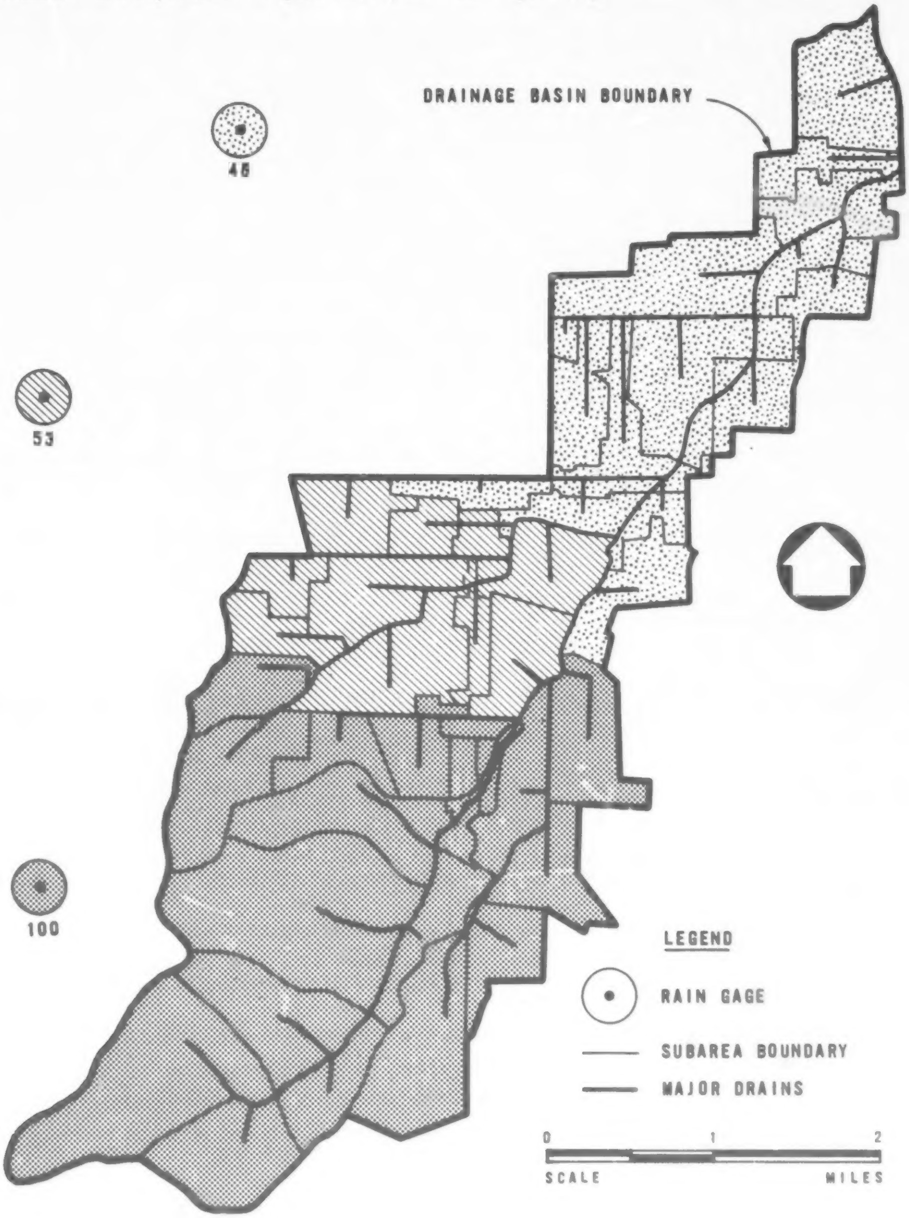

DRAINAGE BASIN BOUNDARY
46
53
100
LEGEND
RAIN GAGE
SUBAREA BOUNDARY
MAJOR DRAINS
0
1
2
SCALE
MILES

Figure 4-11. Subcatchments, drainage network, and rainfall allocations for SWMM simulation.

2. Prepare input data for the runoff block of the EPA SWMM computer program.

Data were obtained from the following sources:

- Topography maps
- Zoning maps
- Sewer and street maps
- County channel cross-section and profile drawings
- Aerial photographs
- Municipality street cleaning records
- Rain gage location maps
- County rainfall records

Numerical input data for each storm were prepared for the following:

a. For the full basin:

- Storm timing, time-step size, time since the previous storm
- Rainfall (three gages)
- Fraction of the impervious area with zero detention
- Street cleaning data
- Parameters to control computer output format

b. For each subcatchment

- Outlet location, area, width, slope, percent imperviousness, roughnesses, surface retention storages, infiltration parameters
- Land use
- Soil erosion parameters
- Channel, pipe, and gutter geometries, roughnesses, lengths, and slopes

3. Gather for each storm the following prototype output data for calibration and verification purposes:

- County streamflow records at the basin outlet (stream gage)
- Water quality data from the analysis of grab samples, collected at the outlet stream gage throughout the storm for BOD, suspended solids, and numerous other constituents

4. Execute the uncalibrated computer program. IBM 370/168 computer requirements per storm were:

	CPU time, min	Total cost, $
Compile and execute	0.50	20.00
Compile only	0.25	--
Execute only	--	7.00

The computed (uncalibrated) outflow hydrograph for the December storm, resulting from the application of the rain gage Station 100 rainfall to the entire basin, is compared with the observed prototype behavior in Figure 4-12.

5. Calibrate the model. This involves adjusting the estimated and uncertain model parameters, and making successive computer runs, until a set of parameters is found that minimizes the total error in both the quantity and quality simulations for all design storms.

The computed outflow hydrographs and pollutographs of both design storms were modified to match the observed behavior as closely as possible by making the following model parameter adjustments:

- Reduce imperviousness to about 70% of their uncalibrated values
- Apply rainfall measured at three different gages (Stations 48, 53, and 100--see Figure 4-12 for hyetographs) to three segments of the basin, as defined by the Thiessen method (see shading in Figure 4-11).
- Reduce fraction of impervious area that is directly connected from 20% to 1%
- Reduce the erosion control practice factor, to reduce erosion
- Adjust the various quality constituent ratios (percentages of suspended solids)

The resulting computed (calibrated) outflow hydrograph for the December storm is compared with the observed and uncalibrated hydrographs in Figure 4-12 .

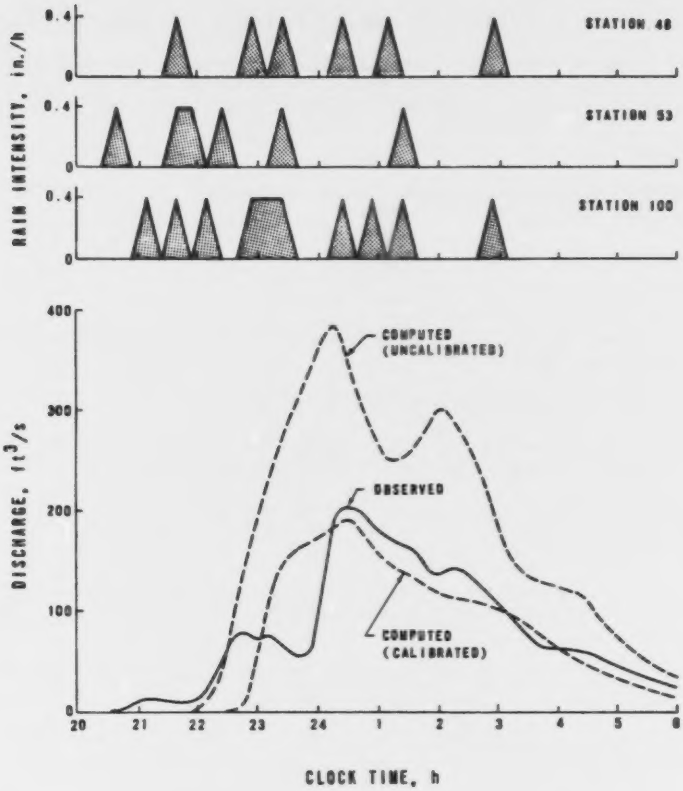


Figure 4-12. Observed rainfall and runoff, and computed runoff hydrographs, for the storm of December 29-30, 1976.

Comments

After completing calibration, as described above, the model must be verified with an additional different storm (or storms).

Note that the 1976-1977 water year storms were selected for technique demonstration only. These storms, occurring in a second consecutive drought year, exhibited abnormally high infiltration-percolation characteristics necessitating the high imperviousness reduction. Cross calibration with normal and wet years would also be required before selecting representative long-term parameter values.

EXAMPLE PROBLEM 4-8: SIZE A STORAGE BASIN FOR THE TEST AREA (EPA SWMM MODEL [7, 28])

Using the synthetic design storm (B1) selected in Example Problem 4-4, determine the storage capacity required to limit BOD released to 50% of the untreated value for this 1 year event. Check the effect of this storage on the other design storms (S1 and S2).

Specified Conditions

1. 15 minute rainfall increments, recorded to the nearest 0.01 in., are defined for the three design storm hyetographs by Figure 4-8.
2. Drainage basin served by separate sewers only.

Assumptions

1. Catchment conditions are the same as those prevailing during the SWMM calibration storms of Example Problem 4-7.
2. The design storms rain uniformly over the entire basin.
3. The storage basin is located at the basin outlet (Figures 4-4, 4-11), separated from the drainage system of Example Problem 4-7 by only a single manhole.
4. The storage basin has a geometric shape, with vertical sides. Outflow is by gravity over a 40 ft long fixed weir at the 10 ft depth level. At the start of the storm the unit is empty.
5. The type of flow within the storage basin is "plug flow" (as opposed to completely mixed). Maximum pollutant removals by sedimentation within storage are: SS 70%, BOD 38.5%, with a decay rate of 0.2/h.
6. Treatment capacity of 1 DWF, or 7.3 Mgal/d (Example Problem 4-3, Step 5a), is available, with secondary treatment efficiency (85% BOD removal).
7. Storage unit sludge is resuspended and ultimately delivered with other trapped solids to the dry-weather treatment plant.

Solution

1. Prepare input data for the Runoff Block of the EPA SWMM computer program. Use the same catchment data as those for the calibrated model of Example Problem 4-7 (Assumption 1). Prepare input rainfall hyetograph data for the uniform application (Assumption 2) of the three design storms of Example Problem 4-4 (Specified Condition 1).
2. Execute the runoff program three times, once for each design storm. IBM 370/168 computer requirements per storm were:

	CPU time, min	Cost, $
Compile	0.25	9.00
Execute	0.75	13.00

   Save the three output files.

3. Prepare input data for the Transport Block of the EPA SWMM computer program. In this case, this consists of specifying only a single manhole (Assumption 3)--the Transport Block is needed to link the Runoff and Storage blocks. Use the quantity and quality output from Runoff (Step 2) as input to Transport.
4. Execute the transport program three times, once for each design storm. IBM 370/168 computer requirements per storm were:

	CPU time, min	Cost, $
Compile	0.40	15.00
Execute	0.04	2.00

5. Select two initial trial storage volumes intended to bracket that which provides a 50% removal efficienty on the B1 design storm.

The total storm BOD load is 9428 lb, from Transport, and its total runoff volume is 95.1 Mgal. About 10 Mgal (7.3 Mgal/d x 1.5 d) will be treated directly, with 85% removal (Assumption 6). The flows simulated by storage/treatment are illustrated in Figure 4-13. With 42.6 Mgal storage capacity (50% of the remaining runoff volume), 85% removal of the trapped BOD and an estimated 30% BOD removal from the overflow by sedimentation (Assumption 5) yields 60% overall BOD removal. Therefore, select 20 and 50 Mgal as initial trial storage volumes.

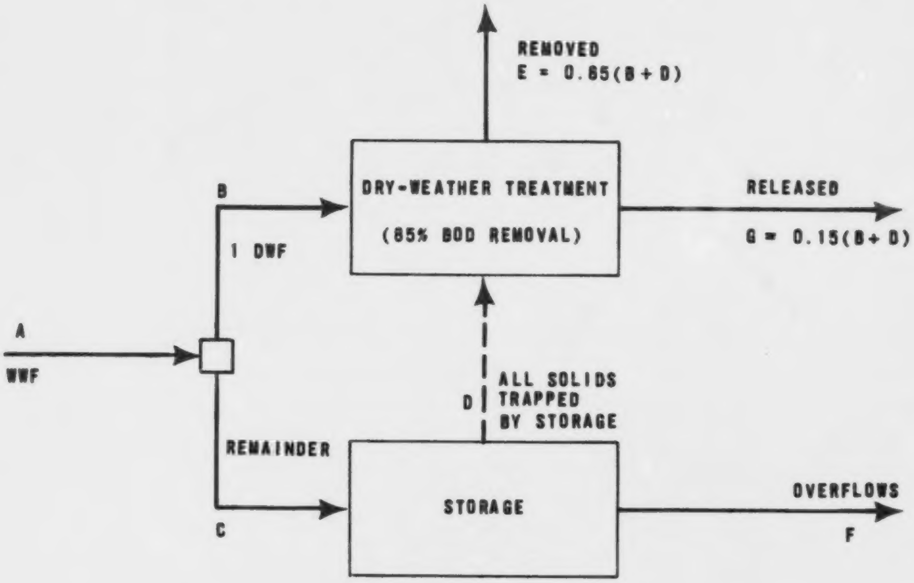


Figure 4-13. Schematic of flows simulated by storage/treatment.

6. Prepare input data for the Storage/Treatment Block of the EPA SWMM computer program. Specify external, in-line storage and treatment with the characteristics prescribed by Assumptions 4 through 6. Specify the base area, computing it from the (assumed) storage volume with a 10 ft sidewater depth. Suppress cost computations. Use the quantity and quality output from Transport (Step 4) as input to Storage/Treatment.

7. Execute the storage/treatment program for each of the storage capacities selected in Step 5, with quantity/quality input from Transport corresponding to the B1 design storm. Computer requirements per run were:

	CPU time, min	Cost, $
Compile	0.40	17.00
Execute	0.04	3.00

8. Compute the BOD releases, as indicated in Figure 4-13 (BOD release = F+G). The zero storage (treatment only) result may also be computed from the Step 7 runs. Plot the results on a graph (heavy dots in Figure 4-14). From this graph, estimate the storage volume that will limit BOD releases from the B1 storm to 50% (4714 lb). Rerun the storage/treatment program to verify this estimate, and repeat as necessary (more heavy dots on Figure 4-14). Result of the two additional trials: 40 Mgal storage capacity limits releases to 4798 lb BOD (50.9%, close enough).

9. Execute the storage/treatment program twice more, with the same 40 Mgal storage capacity, for the input quantity and quality from Transport corresponding to the S1 and S2 design storms. Compute the BOD releases for these two storms in the same manner as in Step 8. The results are compared with the B1 design storm results in Table 4-8.

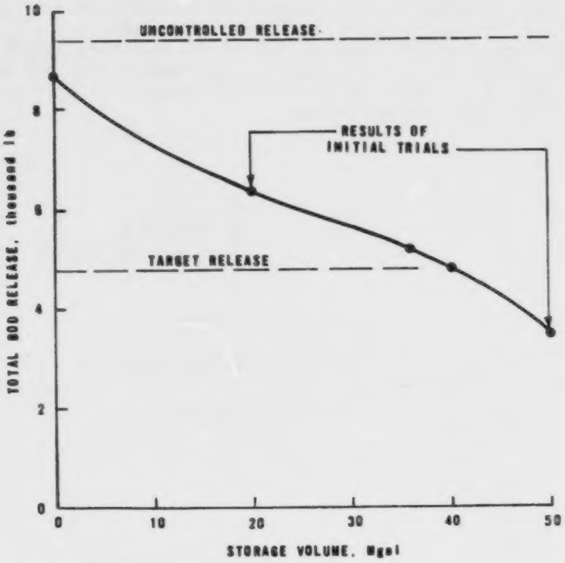


Figure 4-14. Effect of storage volume of BOD discharge of the B1 design storm.

TABLE 4-8. COMPARISON OF BOD RELEASES FROM THREE DESIGN STORMS AS COMPUTED BY SWMM WITH 40 Mgal STORAGE CAPACITY

		Design storm		
		B1	S1	S2
Total precipitation, in.		3.30	2.36	3.18
37.5 hour runoff, in.		0.39	0.29	0.40
BOD inflows, lb				
To treatment	(B)[a]	845	622	730
To storage	(C)	8583	7378	8660
Total	(A)	9428 (100%)	8000 (100%)	9390 (100%)
BOD captured, lb				
By storage	(D)	4602	4901	4954
BOD removals, lb				
Direct treatment	(0.85B)	718	529	621
Treated from storage	(0.85D)	3912	4166	4211
Total	(E)	4630 (49.1%)	4695 (58.7%)	4832 (51.5%)
BOD releases, lb				
Direct treatment	(0.15B)	127	93	109
Treated from storage	(0.15D)	690	735	743
Storage overflow	(F)	3981	1260	3183
Total	(F + G)	4798 (50.9%)	2088 (41.3%)	4035 (48.5%)

a. Refers to flow paths of Figure 4-13.

Comments

1. The low computed runoffs in Table 4-8 (all about 12% of total precipitation) result from the SWMM Runoff Block having been calibrated on storms occurring during an unusually dry winter (Example Problem 4-7).
2. Although the total precipitation of design storm B1 (3.30 in.) is greater than that of S2 (3.18 in.), its runoff is less (0.39 in. versus 0.40 in.). This is due to the differences in rainfall time distribution; with the exception of the brief peak, B1 is far more uniform (Figure 4-8).
3. While a 40 Mgal storage basin would take 5.5 days to pump out and process after the storm through the 7.3 Mgal/d treatment plant, this is deemed acceptable since the design storm magnitude was selected to occur only once a year.
4. The far higher peak of design storm B1 (Figure 4-8) has a significant effect on water quality. High rainfall intensities cause high erosion and scour, resulting in high suspended solids concentrations.
5. The SWMM Storage/Treatment Block computes sedimentation within storage only from that stormwater which overflows. Therefore, the effectiveness of the storage unit as a settling basin cannot be compared with its total capturing capability.
6. The strong impact of the chosen design storm on the required storage capacity, or the BOD removal, is evident from Table 4-8. With constant storage capacity, the BOD removal increases as the storm precipitation decreases.

EXAMPLE PROBLEM 4-9. TEST THE STORAGE BASIN SIZED BY SWMM WITH A LONG HISTORICAL RECORD (METCALF & EDDY CONTINUOUS SIMULATION METHOD, SIMPLIFIED SWMM MODEL [9])

Determine the annual number of overflows, and the pollutant loads discharged, which occur with the storage basin as sized in Example Problem 4-8, over a defined historical record.

Specified Conditions

1. Rainfall period: 1951-1976 (25 years; same as Example Problem 4-6).
2. Rainfall amounts: Daily rainfall increments at San Jose City Hall, increased by 40% (direct extrapolation by isohyetal lines).
3. Drainage basin is served by separate sewers only.
4. Average annual runoff quantity and quality characteristics are the same as those for Example Problem 4-5, Specified Condition 3a, with the exception of the K-factors. These are reduced to 41.4% of the values used in Example Problem 4-5, to reduce the overall runoff coefficient (previously 0.29) to that obtained with SWMM in Example Problem 4-8 (0.12).

Assumptions

1. Same assumptions are made as Assumptions 1 through 3 of Example Problem 4-6.
2. Treatment capacity of one dry-weather flow, or 7.3 Mgal/d, is available (as in Example Problems 4-3, 4-6, 4-8).
3. Treatment removes 85% of each stormwater pollutant; resulting treated effluent concentrations are as per Example Problem 4-6, Assumption 6.
4. Storage basin capacity is 40 Mgal (as sized in Example Problem 4-8).
5. Sedimentation in storage removes 30% of the BOD from overflows (compare with Example Problem 4-8, Assumption 5: 38.5% BOD removal, maximum). Eighty-five percent of the BOD in stormwater captured by storage is removed by subsequent treatment.

Solution

1. Prepare input for the Simplified SWMM computer program, from the data prescribed by the Specified Conditions and Assumption 3, plus the following treatment and storage capacities (Assumptions 2 and 4):

Treatment (withdrawal rate), Mgal/d	Storage capacity, Mgal
0.0	0
7.3	40

2. Execute the program twice, once for each storage/treatment combination. IBM 370/168 computer requirements (execute only) per run were: 0.05 min CPU time, $1.70 total cost.

3. Results for the storage-with-treatment run are summarized in Table 4-9. The uncontrolled release run (zero storage, zero treatment) yielded the following results:

Average annual overflow:	566.2 Mgal
Average annual BOD release:	69 280 lb
Average annual overflow days*:	54.5

TABLE 4-9. SUMMARY RESULT FOR 25 YEARS PERFORMANCE WITH 40 Mgal STORAGE AND 7.3 Mgal/d TREATMENT CAPACITY

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CALABAZAS CREEK SEPARATE SEWERS

WATER YEAR	RAIN IN.	RAIN DAYS	RUNOFF	OVERFLOW Mgal	TREATED Mgal	OVERFL DAYS*	TREAT DAYS	MAX STORAGE Mgal	POLLUTANT LOAD, THOUSANDS OF POUNDS					
									BOD	SS	VSS	TOT N	TOT P	OTHERS
1951	27.34	68	823.1	177.6	645.4	11	88.4	40.0	34.	368.	221.	12.	1.	0.
1952	13.47	56	405.4	2.8	402.6	2	55.2	40.0	8.	86.	51.	6.	0.	0.
1953	14.11	48	424.8	11.3	413.5	5	56.6	40.0	9.	99.	60.	6.	0.	0.
1954	16.59	43	499.4	23.6	475.8	3	65.2	40.0	12.	128.	77.	7.	0.	0.
1955	25.33	64	762.4	198.4	564.0	6	77.3	40.0	35.	379.	228.	11.	1.	0.
1956	14.06	58	423.1	10.8	412.4	2	56.5	40.0	9.	98.	59.	6.	0.	0.
1957	30.67	75	923.4	183.9	739.5	17	101.3	40.0	36.	395.	238.	13.	1.	0.
1958	13.68	37	411.8	89.5	322.2	9	44.1	40.0	17.	185.	111.	6.	1.	0.
1959	14.57	43	438.7	57.5	381.2	5	52.2	40.0	14.	154.	93.	6.	0.	0.
1960	13.85	56	416.8	3.6	413.2	2	56.6	40.0	8.	89.	53.	6.	0.	0.
1961	17.67	45	531.9	140.1	391.8	9	53.7	40.0	24.	266.	160.	8.	1.	0.
1962	23.31	33	701.7	274.9	426.8	8	58.5	40.0	41.	453.	272.	10.	1.	0.
1963	14.24	41	428.6	60.6	368.0	4	50.4	40.0	14.	156.	94.	6.	0.	0.
1964	21.17	63	637.2	102.2	535.0	12	73.3	40.0	22.	245.	147.	9.	1.	0.
1965	10.32	33	310.6	21.9	288.7	3	39.6	40.0	8.	88.	53.	4.	0.	0.
1966	27.71	65	834.0	163.5	670.6	10	91.9	40.0	32.	354.	213.	12.	1.	0.
1967	16.40	50	553.8	127.1	426.7	2	58.4	40.0	23.	256.	154.	8.	1.	0.
1968	29.57	50	890.1	190.8	699.3	14	95.6	40.0	36.	396.	238.	13.	1.	0.
1969	15.85	53	477.1	47.9	429.2	5	58.8	40.0	14.	151.	91.	7.	0.	0.
1970	18.97	53	571.1	152.6	418.5	6	57.3	40.0	26.	288.	173.	8.	1.	0.
1971	8.93	41	268.9	3.3	265.6	2	38.4	40.0	5.	58.	35.	4.	0.	0.
1972	32.09	80	984.1	213.4	770.6	16	105.6	40.0	40.	441.	265.	14.	1.	0.
1973	22.82	72	681.1	14.8	666.2	2	91.3	40.0	14.	155.	93.	10.	0.	0.
1974	17.05	55	513.3	24.3	489.0	4	67.0	40.0	12.	132.	79.	7.	0.	0.
1975	8.06	41	243.2	0.0	243.2	0	33.3	27.4	5.	49.	30.	4.	0.	0.
TOTAL				2296.4		159			498.					
AVERAGE				91 86		6.4			19.92					

*Days with some overflow

4. Adjust computed BOD releases, since Simplified SWMM makes no allowance for sedimentation within storage.

   BOD concentration entering storage (Assumption 3)

   = 2.22 mg/L/0.15
   = 14.8 mg/L (ppm)

   Average annual BOD load removed by sedimentation from 91.86 Mgal/yr average storage overflow (Assumption 5)

   = $0.3\ (91.86 \times 10^6\ \text{gal})\ 8.34\ \text{lb/gal}\ (14.8 \times 10^{-6})$
   = 3402 lb

   Corrected average annual BOD release (Table 4-9)

   = 19 920 - 3402
   = 16 518 lb

5. Compute corrected 25 year BOD removal efficiency

Uncontrolled BOD release = 69 280 lb/yr (Step 3)
Controlled BOD release = 16 518 lb/yr (Step 4)
BOD removal = 52 762 lb/yr
= 76.2%

Comments

1. Given the stated assumptions, the 40 Mgal storage basin, combined with 1 DWF treatment capacity, reduces the number of overflows by 88% (6.4 versus 54.4 days), and removes 76% of the BOD load, over the long term. This latter figure compares with a 49% BOD removal for the 1 year design storm event (Example Problem 4-8, Table 4-8), and is 90% of the defined optimal treatment efficiency (85%).

There are two primary observations that may be made from the foregoing series of examples:

1. The range of applications for which such models may be used is very broad.

2. The capabilities of the models in their original forms to be able to serve the required purposes are notably limited in most cases.

The tremendous range of possible model applications has only been hinted at by these example problems.

While the models are of great help, they usually only partially fulfill the task requirements. Often, additions or modifications must be patched in; these are easier to effect in the more flexible desktop models, but they are presently less complete and tested. Model results usually must be interpreted and often adapted. All these considerations, together with the more fundamental question of model applicability, serve to underscore the need for appropriately qualified and experienced professionals to oversee usage.

Further, more specific observations that may be made about the preceding Example Problems are:

- The URS desktop procedure of Example Problem 4-2 requires much tedious hand computation, and for the amount of effort involved offers low accuracy by comparison with a detail event model such as SWMM.

- While a detailed event model provides excellent detail (Example Problem 4-8), it requires a significant investment in prior calibration (Example Problem 4-7).

- The simplified, continuous simulation models offer relatively high benefits for low costs and effort (Example Problems 4-5 and 4-6). They make possible good inexpensive assessments of the long-term impacts of designs (Example Problem 4-9).

- The economic procedures (Example Problem 4-3) are untested in applications. They need a fair testing period, with further shakedown and evolution. They appear to be particularly sensitive to unit costs, which should be investigated further.
- The selection of design storms (Example Problem 4-4) can be a tricky business, with significant consequences Some standardization of procedures for the various prevailing conditions would be desirable.

SECTION 5

STORMWATER CHARACTERISTICS - DATA BASE AND NORMALIZATION

In order to address a stormwater runoff or combined sewer overflow problem, an investigator must have knowledge of the characteristics of the problem. This section presents an overview of four areas that are integral to the solution of urban runoff problems.

1. Sources of Stormwater Pollutants. Stormwater pollutants are materials washed from the air and the land surface during rainfall or snowmelt events. It is reasonable to assume that some land surface activities, uses, and characteristics will cause greater pollutant loading than others. Known causal relations will be examined and quantified if possible. An understanding of sources allows some estimation of loadings, pinpoints areas that require in-depth survey, and provides the basis for developing nonstructural control alternatives.

2. Discharge Characteristics. Data gathered from several studies of stormwater runoff and combined sewer overflow are presented as a guide to what can be expected at the "end of the pipe." The information gives the investigator a starting point with which to compare stormwater pollutants to other sources within a basin and evaluate site specific data for its applicability.

3. Residuals. Solids derived from stormwater treatment must be considered in developing a complete pollution abatement program. It is necessary to evaluate the anticipated quantities and characteristics in order to provide for the final disposal of the treatment sludges.

4. Receiving Water Impacts. The goal of any stormwater study is the mitigation or prevention of adverse impacts on the receiving water. Summaries of studies of stormwater pollution impacts will be presented. The data indicate the potential adverse effects and some approaches to the evaluation of impact.

SOURCES OF STORMWATER POLLUTANTS

An understanding of the potential sources of stormwater pollutants is of primary importance when studying the impact of urban runoff. The accumulation of the various pollutants within a basin can be attributed to several sources and the individual effects are difficult to separate. However, a qualitative knowledge of the probable sources enables an investigator to concentrate on expected problem areas and evaluate source controls that could be used to curtail an adverse pollutant loading before it reaches the sewer system. The principal sources of runoff pollutants are as follows [1]:

1. Street pavement. The components of road surface degradation can become part of the urban runoff loading. The aggregate material is the largest contributor and additional quantities will come from the binder, fillers, and any substance applied to the surface. The amount of pollutants will depend on the age and type of surface, the climate, and the quantity and type of traffic.

2. Motor vehicles. Vehicles can contribute a wide variety of materials to the street surface runoff. Fuels and lubricants spill or leak, particles are worn from tires or brake linings, exhaust emissions collect on the road surface, and corrosion products or broken parts fall from vehicles. While the quantity of material deposited by motor vehicles is expected to be relatively small, the pollution potential is important. Vehicles are the principal nonpoint source of asbestos and some heavy metals including lead.

3. Atmospheric fallout. Air pollutants include dust, contaminants, and particles from industrial stacks and vents, from automobiles and planes, and from exposed land. The airborne matter will settle on the land surface and washoff as contaminated runoff. The potential significance of dustfall was indicated during a study done in Cincinnati [2]. During the study period 567 kg/ha (506 lb/acre) of dustfall were measured at a monitoring station and 818 kg/ha (730 lb/acre) of suspended solids were measured in storm runoff.

4. Vegetation. Leaves, grass, clippings, and other plant materials that fall or are deposited on urban land will become part of the runoff problem. Quantities will depend on the geographic location, season, landscaping practices, and disposal methods.

5. Land surface. The type of ground cover found in a drainage basin and the amount of vehicular and pedestrian traffic is a function of land use and will affect the quality of storm runoff.

6. Litter. Litter consists of various kinds of discarded refuse items, packaging material, and animal droppings. Although the quantities are small and not significant sources of pollution, the debris is highly visible in a receiving stream and can be a focal point for citizen complaints.

7. Spills. These obvious surface contaminants can include almost any substance hauled over city streets. Dirt, sand, and gravel are the most common examples. Industrial and chemical spills are potentially the most serious.

8. Anti-skid compounds and chemicals. Cold weather cities employ large amounts of substances designed to melt ice during the winter. Salts, sand, and ash are the commonly used agents. A variety of other chemicals may be used as fertilizers, pesticides, and

herbicides. Most of these materials will become part of the urban runoff.

9. Construction sites. Soil erosion from land disturbed by construction is a highly visible source of solids in storm runoff. Important urban sites will include large scale projects such as highway construction and urban renewal. The construction methods and control measures will influence quantities.

10. Collection network. Storm sewer networks using natural or improved earthen channels will be subject to erosion of the banks. Collection networks also tend to accumulate deposits of material that will be dislodged and transported by storm flows.

It is obvious from this list that there are many potential sources of pollutants within each basin and the sources vary in importance. The quantities that accumulate are a function of natural conditions and urban development. Most of the sources exist concurrently in the urban environment and, although their effects cannot be isolated, some relative quantities are discussed in the following sections.

Street Pavement

Several studies of pavement wear in Germany [3] have indicated that at least 0.05 cm (0.02 in.) of surface will be worn from a tire lane during a summer. Assuming four tire lanes each 1 m (3 ft) wide in a 7.5 m (24 ft) road, this wear would amount to 0.66 kg/m^2 (0.13 lb/ft^2) of road per summer. The wear in the winter can be considerably greater if studded snow tires are used by a large portion of the cars. The effect is shown in Table 10 for northern and southern Germany; the southern locations are in the Alps region where 30 to 40% of the cars use studded tires.

TABLE 10. ABRASION OF ASPHALT-CONCRETE HIGHWAY SURFACES IN WINTER AND SUMMER

Site	Vehicles per 24 hours	Abrasion, in.	
		Summer	Winter
Northern Germany	7 500	0.01-0.03	0.02-0.04
	4 500	0.004-0.01	0.02
Southern Germany	5 000	0.02-0.03	0.06-0.07
	9 000	0.02-0.04	0.07-0.10
	18 000	0.02	0.23-0.26

in. x 2.54 = cm

Motor Vehicles

A detailed study of street surface pollutants in Washington, D.C., found that most of the contaminants were traffic related [4]. This does not mean that the pollutants necessarily originate with the vehicle itself but rather that the expected loading intensity can be expressed in the form:

$$Y = B + mX \qquad (5\text{-}1)$$

where Y = loading intensity, kg/mi (lb/mi)
B = amount of pollutant unrelated to traffic, kg/mi (lb/mi)
m = traffic related deposition rate, kg/axle·km (lb/axle·mi)
X = traffic in axles.

The values of m, deposition rate, for traffic related contaminants are shown in Table 11. Depositions of orthophosphate, fecal coliforms, fecal streptococci, cadmium, PCBs, and litter were not considered to be traffic related. The values of B for both asbestos and lead were negative, indicating that these important pollutants are entirely traffic related.

TABLE 11. DEPOSITION RATES OF TRAFFIC-RELATED ROADWAY MATERIAL [4]

Parameter	Units	Deposition rate
Dry weight	lb/1000 axle-mi	2.38
Volume	qt/1000 axle-mi	0.63
Volatile solids	lb/1000 axle-mi	0.12
BOD	↕	5.43×10^{-3}
COD		0.13
Grease		1.52×10^{-2}
Total phosphate - P		1.44×10^{-3}
Nitrate - N		1.89×10^{-4}
Nitrite - N		2.26×10^{-5}
Kjeldahl - N		3.72×10^{-4}
Chloride		2.20×10^{-3}
Petroleum		8.52×10^{-3}
n-paraffins	lb/1000 axle-mi	5.99×10^{-3}
Asbestos	fibers/axle-mi	$3.86 \times 10^{+5}$
Rubber	lb/1000 axle-mi	1.24×10^{-2}
Lead	↕	2.79×10^{-2}
Chromium		1.85×10^{-4}
Copper		2.84×10^{-4}
Nickel		4.40×10^{-4}
Zinc		3.50×10^{-3}
Magnetic fraction	lb/1000 axle-mi	0.13

lb/1000 axle-mi x 0.28 = kg/1000 axle-km
qt/1000 axle-mi x 0.59 = L/1000 axle-km

Although only a small fraction of the traffic related deposits come directly from vehicles, it is an important fraction. Grease, petroleum, lead, zinc, copper, nickel, chromium, and asbestos are all potentially toxic to aquatic life and all originate directly from vehicles. The remaining traffic related organics, nutrients, and solids are products of road surface abrasion or have been carried to the roadway by vehicular action.

The values for pollutant deposition shown in Table 11 were developed by sweeping and washing sections of street at 24 hour intervals. Additional samples, taken to compute accumulation of material for a 3 day interval, showed that accumulation is not a linear function of the deposition rate. The Washington, D.C., work showed that roadway accumulation levels off in about 4 days due to traffic related removal mechanisms. Although dust and dirt are blown onto adjacent land surfaces by vehicle movement and other means, at least a portion of the displaced material is still available for transport by storm runoff.

A calculation of tire wear in a German study [3] indicates that the weight loss per tire is 12% or 0.9 kg (2 lb) over a lifespan of 30 000 km (20 000 mi). Therefore, the potential deposition rate for four tire vehicles is 0.12 kg/km (0.4 lb/mi) per 1 000 vehicles. The tire rubber consists of 87% carbon, 6% hydrogen, 2% sulfur, and 2% zinc oxide.

Vegetation

Waste vegetative matter is an important source of organic and nutrient pollutants in urban stormwater. The quantity of leaves, grasses, seeds, and clippings will depend upon the particular urban area and public works practices. Vegetative waste will become part of the urban runoff when material falls or is dumped onto impervious areas and when pollutants are leached from decaying organic matter.

Typical concentrations of nutrients in vegetative litter are shown in Table 12.

TABLE 12. NUTRIENTS IN VEGETATIVE LITTER [5]
Percentage, dry weight

	Nitrogen	Phosphorus	Potassium	Ash
Evergreen leaves	0.58-1.25	0.04-0.10	0.12-0.39	3.01-4.33
Deciduous leaves	0.51-1.01	0.09-0.28	0.40-1.18	5.71-15.16

Studies of quantities of waste vegetative matter have generally been performed by scientists interested in forest ecosystems. Consequently, quantitative estimates deal with full canopy situations. Estimates for urban areas should be modified to account for lower tree densities; quantitative estimates are presented in Table 13.

TABLE 13. VEGETATIVE LITTER PRODUCTION [6]
lb/acre·yr

Source	Yield of waste matter
Evergreens[a]	3300
Deciduous trees[a]	2854
Rye grass[b]	3675-5612

a. Full canopy.

b. Florida.

lb/acre·yr x 1.121 = kg/ha·yr

Land Surface

General land use categories are an important basis for studying stormwater pollution because of the relation between land use and many specific sources. For example, there is usually less dustfall in a residential-commercial area than in an industrial zone and there is heavier motor vehicle traffic in a commercial-industrial area than in residential neighborhoods. In this sense, evaluation of pollutants versus surface use will include two hard to quantify sources--litter and spills.

Three major research studies have documented the effects of land use on the accumulation of pollutants in urban areas [4, 7, 8]. While the reports are not directly comparable with each other because of different collection and analyzing techniques, they show the relative influence of land use. A summary of the studies is shown in Table 14.

The specific pollutants found in urban runoff will be affected also by the different categories of land use. The differences are shown in Table 15.

The data in Tables 14 and 15 were obtained by sweeping, vacuuming, or washing pollutants from street surfaces in urban areas with the specific land use noted. The areas sampled were small enough to be a valid indication of the differences in pollutant accumulation for general land uses. However, the dry street surface samples do not necessarily represent the portion that will wash off during a runoff event and do not include pollutant loadings from areas other than streets.

Anti-Skid Compounds and Chemicals

It is difficult to quantify chemicals that are a source of stormwater pollutants because of great variations in application rates. A few ranges can be presented as an indication of the potential magnitude of the problem.

Salt application for deicing can be a serious source of chlorides in runoff; ranges of application rates are shown in Table 16.

TABLE 14. DUST AND DIRT ACCUMULATION RATES FOR DIFFERENT LAND USES

	Single family	Multi-family	Commercial	Industrial
APWA at Chicago [7]				
Mean, lb/curb-mi·d	37	121	174	284
Median, lb/curb-mi·d	18	90	143	111
Number of samples	60	93	126	46
Adjusted URS data at several cities [8]				
Mean, lb/curb-mi·d	155	107	46	292
Mean without extreme, lb/curb-mi·d	71	56	20	138
Median, lb/curb-mi·d	69	32	20	74
Number of samples	21	14	17	20
Biospherics at Washington, D.C., (shopping center only) [4]				
Mean, lb/curb-mi·d	...	...	62	...
Median, lb/curb-mi·d	...	...	67	...
Number of samples	...	...	8	...
Overall mean, lb/curb-mi·d	45	110	150	240

lb/curb-mi·d x 0.28 = kg/curb km·d

Abrasives used on street surfaces will also become part of direct runoff or snowmelt runoff in proportion to the amount applied. Stockpiles of salt or abrasives may also be important point sources of pollutants.

The next most important source of chemicals is the application of fertilizers, insecticides, and herbicides. Although quantities are small, the enrichment or toxic effects make them important to runoff studies. In a multicity study performed in 1971, quantities of pesticides were measured in road dust. Presumably, this is material that will easily wash off into receiving waters during a runoff event; ranges are given in Table 17.

Construction Sites and Collection Networks

The principal mechanism of pollution from these two sources is erosion. Soil erosion is a major source of stormwater solids for urban and suburban areas. The problem areas are construction sites, undeveloped areas, highway cuts, urban renewal areas, and drainage ditches themselves. In addition to specific sources, general erosion will take place from all unpaved areas. Erosion is a function of a number of physical conditions, and it is difficult to predict an erosion quantity for a complete urban area; however, an understanding of the mechanism of erosion is important when considering potential management techniques.

TABLE 15. CONCENTRATIONS OF POLLUTANTS BY LAND USE CHARACTERISTICS [6]
ppm of Dry Solids Unless Otherwise Noted

	Land use			
Pollutant	Single family residential	Multifamily residential	Commercial	Industrial
BOD	5 260	3 370	7 190	2 920
COD	39 300	42 000	61 700	25 100
Total nitrogen	460	550	420	430
Soluble PO_4-P	16	19	20	8
Cadmium	3.3	2.7	2.9	3.6
Chromium	200	180	140	240
Copper	91	73	95	87
Iron	21 300	18 500	21 600	22 500
Manganese	450	340	380	430
Nickel	38	18	94	44
Lead	1 570	1 980	2 330	1 590
Strontium	32	19	17	13
Zinc	310	280	690	280
Fecal coliforms, No./g	82 500	388 000	36 900	30 700
Total coliforms, No./g	891 000	1 900 000	1 000 000	419 000

TABLE 16. SALT APPLICATION FOR DEICING [9]

Area	Application rate per snowday, lb/mi
Northeastern states	670-1 820
North-central states	10-1 840
Southern states	0-1 610
West-central states	110-550
Southwestern states	300-400
Western states	0-1 320

lb/mi x 0.28 = kg/km

TABLE 17. PESTICIDE LOADS FOUND IN SEVERAL CITIES [1]

Pesticide	Lb/curb-mi Range	 Median value
Dieldrin	3-27	24
PCB	65-3 400	1 100
BP-DDD	0.5-120	67
Methoxychlor	0-8 500	.....
P, P-DDT	1-170	61
Endrin	0-2	.....
Methyl parathion	0-20	.....
Lindane	0-17	.....
Total pesticides	136-11 910	1 420

lb/curb-mi x 0.28 = kg/curb-km

The Universal Soil Loss Equation is an empirical formula derived by the Agricultural Research Service to estimate average annual erosion from farm plots. Since it was statistically developed to estimate gross erosion from small areas over a period of years, it is more of a management tool than a predictive formula. The equation is:

$$A = R \cdot K \cdot LS \cdot C \cdot P \qquad (5\text{-}2)$$

where A = soil loss, mass/unit area
R = rainfall factor
K = soil erodibility factor
LS = slope length gradient factor
C = ground cover index factor
P = erosion control factor

The factor R accounts for rainfall energy and intensity, K considers the ease with which the particular soil can be eroded, and LS is a function of slope length and gradient. The factors C and P are the keys to the control of erosion since they are more easily modified than the other three factors. Both were empirically developed by assuming that loose, noncompacted soil with no cover represents C and P factors of 1.0. The use of cover material or erosion control practices will reduce the factors and the amount of soil loss. Representative values are shown in Tables 18 and 19.

Examples of typical erosion rates are shown in Table 20. The quantities indicate a substantial increase in erosion when land is developed for either agriculture or urbanization. The particularly heavy rates from construction activities point out the need to apply control technology to urban and highway construction sites.

TABLE 18. GROUND COVER FACTOR "C" [5]

Type of cover	C value
None	1.0
Permanent seeding	
First 60 days	0.40
60 days to 1 year	0.05
After 1 year	0.01
Sod	0.01
Hay or straw	
1.0 ton/acre	0.20
2.0 tons/acre	0.05
Stone or gravel	
15 tons/acre	0.80
60 tons/acre	0.20
Chemical mulch (90 days)	0.50
Woodchips	
2 tons/acre	0.80
7 tons/acre	0.20

tons/acre x 2240 = kg/ha

Summary

Many sources of stormwater pollutants are present in a basin and their effects interact and overlap. It is difficult to attribute the pollutants measured at the discharge from a basin to a specific source within the drainage area. The importance of this section is in understanding why there is a problem and why construction site erosion prevention should be practiced or why the drainage from a highway intersection should be diverted from a sensitive stream. Results of studies giving overall pollutant concentrations follow.

DISCHARGE CHARACTERISTICS

The investigation of stormwater discharges is concerned with two different types of polluted flows--separate stormwater runoff from storm sewers or drainage channels and combined sewer overflows from sewers containing both runoff and sanitary sewage. The sources of runoff contamination have been described in the preceding section and it is evident that surface runoff has the potential to transport a significant load of pollutants. In this section the results of several monitoring efforts will be presented to indicate the range of pollutant concentrations that can be expected. Some explanation of the individual studies is given to help the reader judge the applicability of the data to his particular problem.

TABLE 19. EROSION CONTROL FACTOR "P" [5]

Surface condition	P value
Loose as a disced plow layer	1.0
Compact, smooth, scraped up, and downhill	1.3
Compact, raked up, and downhill	1.2
Compact, smooth, scraped across slope	1.2
Compact, raked across slope	0.9
Rough, irregular surface	0.9
Loose with rough surface	0.8
Loose with smooth surface	0.9
Structures	
Sediment basin	
0.04 basin/acre	0.5
0.06 basin/acre	0.3
Downstream sediment basin	
With chemicals	0.1
Without chemicals	0.2

acre x 0.405 = ha

TABLE 20. EROSION RATES [5]

Sediment source	Erosion rate, tons/mi^2·yr	Geographic location	Comment
Natural	15-320		
Agricultural	200-70 000		
Urban	50,000	Kensington, Md.	Extensive construction
	1 000-100 000		Small urban construction area
	1 000	Washington, D.C.	750 mi^2 average
	500	Philadelphia, Pa.	
	146-2 300	Washington, D.C. watersheds	As urbanization increases
Highway construction	36 000	Fairfax Co., Va.	Construction on 179 acres
	50 000-150 000	Georgia	Cut slopes

tons/mi^2·yr x 3.5 = kg/ha·yr
mi^2 x 2.590 = km^2

Urban Stormwater Runoff

The quality of urban runoff has been investigated at several sites across the country. The techniques, methodology, and goals varied from project to project, but the combined results present a good indication of the concentrations of pollutants that can be expected in urban runoff. The results of several representative sampling efforts are shown in Table 21. The samples were taken in various parts of the country, from diverse land use, during different seasons, and during dissimilar rainfall events. The average pollutant concentrations shown in the table indicate an order of magnitude of the stormwater runoff problem and the ranges indicate the wide variations in concentrations that may be anticipated. The individual studies involved will show some of the relationships between runoff quality and land or storm characteristics.

TABLE 21. POLLUTANT CONCENTRATIONS IN STORMWATER RUNOFF

	Average pollutant concentrations, mg/L									
City	TSS	VSS	BOD	COD	Kjeldahl nitrogen	Total nitrogen	Phos-phorus	OPO_4-P	Lead	Fecal coliforms[a]
Atlanta, Georgia [10]	287	...	9	48	0.57	0.82	0.33	....	0.15	6 300
Des Moines, Iowa [11]	419	104	56	...	2.09	3.19	0.56	0.15	....	......
Durham, North Carolina [12]	1 223	122	..	170	0.96	....	0.82	....	0.46	230
Knoxville, Tennessee [13]	440	...	7	98	1.9	2.5	0.63	0.30	0.17	20 300
Oklahoma City, Oklahoma	147	...	22	116	2.08	3.22	1.00	1.00	0.24	40 000
Tulsa, Oklahoma [14]	367	...	12	86	0.85	....	....	0.38	....	420
Santa Clara, California	284	70	20	147	....	5.8	0.23	....	0.75	......
Pullach, Germany [3]	158	53	11	125	....	...	....	....	....	......
Average (not weighted)	415	88	20	113	1.41	3.11	0.62	0.46	0.35	13 500
Range	147-1 223	53-122	7-56	48-170	0.57-2.09	0.82-5.8	0.33-1.00	0.15-1.00	0.15-0.75	230-40 000

a. Organisms/100 mL.

Atlanta, Georgia--

The purpose of the Atlanta study was to investigate the impact of urban runoff and combined sewer overflows on the Chattahoochee River, a major water supply and recreational river in the Southeast. Samples were taken from storm runoff at four suburban sites and one downtown location in order to calibrate the runoff model being used to estimate pollutant loading. The results of the sample analysis are shown in Table 22. The four suburban areas vary in land use characteristics as shown in Table 23.

It is difficult to draw detailed conclusions from the limited number of samples taken in Atlanta, but the authors listed some comparisons that may be valid indications of pollutional trends [10].

- The downtown sample is far more heavily polluted than the suburban samples. For most pollutant concentrations measured, the downtown sample is greater than any suburban sample.

- The suspended solids and COD concentrations increase as the percent of the basin that is developed increases.
- Increased lead concentrations appear to be linked with increased commercial land use; this is probably due to large traffic volumes in shopping areas.
- BOD, phosphorus, and total nitrogen did not appear to be related to land use.

TABLE 22. POLLUTANT CONCENTRATIONS IN STORMWATER RUNOFF, ATLANTA, GEORGIA [10]

	Pollutants							
Site	TSS	BOD	COD	Kjeldahl nitrogen	Total nitrogen	Phosphorus	Lead	Fecal coliforms[a]
Montreal Road								
Mean, mg/L	215	6	26	0.73	0.94	0.33	0.08	11 000
Parkside Circle								
Mean, mg/L	296	11	61	0.52	0.72	0.31	0.21	4 600
Plantation Lane								
Mean, mg/L	323	12	63	0.57	0.83	0.35	0.27	2 100
Drew Valley Road								
Mean, mg/L	428	9	71	0.53	0.91	0.35	0.13	6 500
Suburban Total								
Mean, mg/L	287	9	48	0.57	0.82	0.33	0.15	6 300
Range	1-1 989	0-42	5-164	0.25-1.06	0.38-1.51	0.01-1.28	0.05-0.8	10-104 000
No. of samples	63	60	60	13	8	60	59	53
Downtown sample	277	76	597	1.53	2.45	0.37	2.20	......

a. Organisms/100 mL.

TABLE 23. LAND USE CHARACTERISTICS, ATLANTA, GEORGIA

		Land use, %			
Site	Density people/acre	Residential	Commercial	Industrial	Open
Montreal Road	3.1	36	6	18	40
Parkside Circle	5.6	53	15	3	29
Plantation Lane	5.1	14	18	58	
Drew Valley Road	2.8	90	0	10	

people/acre x 2.47 = people/ha

Des Moines, Iowa--

The Des Moines study was an evaluation of potential solutions to stormwater runoff problems and included a sampling program to analyze the quality of combined sewer overflows, storm runoff, and the receiving waters. The pollutant concentrations found in runoff from three areas with separate sewer systems are shown in Table 24. The values indicate that there is very little difference in average quality between the three areas. The sampling program covered both winter and summer runoff conditions with snowmelt as well as direct runoff. A comparison of snowmelt runoff versus rainfall runoff is shown in Table 25. The data indicate that phosphorus is the only pollutant showing a significant effect due to the form of precipitation. The investigators in Des Moines also found that pollutant concentrations generally decreased with time during a storm and cumulative pollutant loading usually "ran ahead" of cumulative flow quantities. These patterns were attributed to a first flush effect in which loose surface material is suspended by the initial runoff water, making it more concentrated than runoff later in the storm.

TABLE 24. POLLUTANT CONCENTRATIONS IN STORMWATER RUNOFF, DES MOINES [11]
mg/L

Site	TSS	VSS	BOD	NH_3-N	Total nitrogen	PO_4-P	OPO_4-P	Comments
S-1, mean	315	99	48	1.99	3.10	0.41	0.06	7.4 people per acre; older residential with considerable park space
S-3, mean	578	101	63	1.60	3.07	0.33	0.14	5.3 people per acre; residential with considerable grassy area
0-11, mean	404	110	56	2.30	3.24	0.70	0.19	10.7 people per acre; considerable industrial and commercial
Summary								
Mean	419	104	56	2.09	3.19	0.56	0.15	
Range	9-3 170	6-484	12-166	0-27.8	0-29.7	0-3.92	0-2.36	
No. of samples	89	84	84	49	49	36	45	

acre x 0.405 = ha

TABLE 25. COMPARISON OF RUNOFF QUALITY FOR SNOWMELT VERSUS RAINFALL [1]

	Pollutants						
	TSS	VSS	BOD	NH_3-N	Total nitrogen	PO_4-P	OPO_4-P
Rainfall runoff							
Mean, mg/L	426	96	51	2.21	3.25	0.68	0.19
No. of samples	48	45	56	31	31	27	32
Snow melt runoff							
Mean, mg/L	411	113	65	1.89	3.08	0.22	0.04
No. of samples	41	39	28	18	18	9	13

Durham, North Carolina--

The Durham study was not designed to be as site specific as the previous two studies in that variables affecting runoff quality were analyzed to develop predictive equations. Although the data were based on samples from the Durham area, the form of equations and relationships between variables and loading should be applicable to other areas with similar climate and topography. Many pollutants were analyzed during the study and the mean values are shown in Table 26. Regression analysis was performed to relate pollutant loading to four variables considered to have important effects on runoff quality. The four variables were rate of runoff, time from storm start, time from last storm, and time from last peak. The first two variables dealing with the storm event were found to be the most influential and little correlation increase resulted from considering elapsed time between storms or peaks. The final regression equations are shown in Table 27; CFS is the runoff quantity in cubic feet per second and TFSS is the elapsed time from the storm start.

TABLE 26. POLLUTANT CONCENTRATIONS IN STORMWATER RUNOFF, DURHAM, NORTH CAROLINA [12]

Pollutant	Mean, mg/L	Range, mg/L	No. of samples
COD	170	20-1 042	491
TOC	42	5.5-384	413
Total solids	1 440	194-8 620	325
Volatile solids	205	33-1 170	221
TSS	1 223	27-7 340	408
VSS	122	5-970	312
Kjeldahl nitrogen, N	0.96	0.1-11.6	313
Total phosphorus, P	0.82	0.2-16	310
Fecal coliforms (No./mL)	230	1-2 000	327
Aluminum	16	6-35.7	63
Calcium	4.8	1.1-31	180
Cobalt	0.16	0.04-0.47	145
Chromium	0.23	0.06-0.47	232
Copper	0.15	0.04-0.50	225
Iron	12	1.3-58.7	257
Lead	0.46	0.1-2.86	336
Magnesium	10	3.6-24	217
Manganese	0.67	0.12-3.2	244
Nickel	0.15	0.09-0.29	103
Zinc	0.36	0.09-4.6	310
Alkalinity	56	24-124	80
BOD[a]	60	2-320	208

[a] The authors feel that BOD results were affected by changing dilutions in the laboratory and recommend that BOD not be considered an appropriate measure of pollutant strength. (See p. 48, Reference [12] for the full discussion.)

TABLE 27. REGRESSION EQUATIONS RELATING POLLUTANT CONCENTRATION TO RUNOFF CHARACTERISTICS, DURHAM, NORTH CAROLINA [12]

Pollutant	Concentration, mg/L	
	As developed	Normalized
TSS	$222\ \mathrm{CFS}^{0.23}\ \mathrm{TFSS}^{-0.16}$	$1102\ \mathrm{R}^{0.23}\ \mathrm{TFSS}^{-0.16}$
VSS	$44\ \mathrm{CFS}^{0.18}\ \mathrm{TFSS}^{-0.17}$	$153\ \mathrm{R}^{0.18}\ \mathrm{TFSS}^{-0.17}$
COD	$113\ \mathrm{CFS}^{0.11}\ \mathrm{TFSS}^{-0.28}$	$242\ \mathrm{R}^{0.11}\ \mathrm{TFSS}^{-0.28}$
Kjeldahl nitrogen	$0.85\ \mathrm{CFS}^{0.87}\ \mathrm{TFSS}^{-0.29}$	$363\ \mathrm{R}^{0.87}\ \mathrm{TFSS}^{-0.29}$
Total phosphorus	$0.80\ \mathrm{CFS}^{0.03}\ \mathrm{TFSS}^{-0.29}$	$0.98\ \mathrm{R}^{0.03}\ \mathrm{TFSS}^{-0.29}$
Lead	$0.27\ \mathrm{CFS}^{0.125}\ \mathrm{TFSS}^{-0.29}$	$0.64\ \mathrm{R}^{0.125}\ \mathrm{TFSS}^{-0.29}$

Note: CFS = runoff, ft^3/s
TFSS = time from storm start, hours
R = runoff, in./h

$\mathrm{ft}^3/\mathrm{s} \times 0.028 = \mathrm{m}^3/\mathrm{s}$
in./h x 2.54 = cm/h

The quantity is necessarily dependent on the area of the Durham basin, 417 ha (1029 acres), and so the equations cannot be directly compared with results from other sites. In the second set of equations in Table 27, the equations have been normalized by converting cubic feet per second of runoff to inches per hour of runoff using the actual area of the basin. In most cases the pollutant concentrations increase with greater quantities of runoff, indicating increased erosion, pickup, and transport capacities of higher flows. The concentrations also tend to decrease as a storm event continues, indicating that the reservoir of pollutants on the land surface decreases or at least becomes more difficult to pick up and transport.

Knoxville, Tennessee--

The purpose of the Knoxville study was to investigate the effects of urbanization on an area of Tennessee that overlies a formation of soluble carbonate rock. The principal concern was that urbanization would greatly increase the impervious fraction of a basin and consequently cause increased runoff quantity with the associated pollutant loading. During the investigation, samples were taken from four urbanizing watersheds, upstream areas, and precipitation in an effort to determine probable impacts. The data obtained from the project watersheds are presented in Table 28. An interesting analysis made in Knoxville was the comparison of atmospheric input to a basin (dry fallout and precipitation) and output (streamflow); the analysis is shown in Table 29. Fourth Creek, First Creek, and Plantation Hills the streamflow is mostly storm runoff and it is shown that atmospheric sources are particularly important.

TABLE 28. POLLUTANT CONCENTRATIONS IN STORMWATER RUNOFF, KNOXVILLE, TENNESSEE [13]

Site	Pollutants, mg/L									
	TSS	BOD	COD	Kjeldahl nitrogen	NO_3-N	PO_4-P	OPO_4-P	Lead	Mercury	Fecal coliforms[a]
Fourth Creek, mean	1 200	12	110	2.4	0.7	1.1	0.20	0.34	0.0026	....
Third Creek, mean	240	9.1	95	1.5	0.6	0.49	0.26	0.13	0.0004	....
First Creek, mean	150	6.3	32	0.65	0.6	0.56	0.46	0.13	0.0006	....
Plantation Hills, mean	46	2	29	1.0	0.4	0.36	0.32	0.08	0.0014	20 300
Total										
Mean	440	7.4	98	1.9	0.6	0.63	0.30	0.17	0.0017	20 300
Range	3-6 400	0-86	12-700	0.04-13	0.01-12	0.03-6.9	0.01-1.6	0-1.6	0.00005-0.047	670-700 000
No. of samples	175	181	70	76	177	183	176	189	76	40

Note: Fourth Creek - 0.82 acres, 45% impervious, commercial.
Third Creek - 1.60 acres, 28% impervious, industrial-residential.
First Creek - 0.5 acre, 16% impervious, residential.
Plantation Hills - 0.24 acre, 23% impervious, suburban.

a. Organisms/100 mL.

acre x 0.405 = ha

TABLE 29. COMPARISON OF WATERSHED LOADINGS, ATMOSPHERIC INPUT VERSUS RUNOFF OUTPUT, KNOXVILLE, TENNESSEE [13]

Site	Annual loading, lb/acre					
	TSS	COD	Kjeldahl nitrogen	NO_3-N	PO_4-P	Lead
Fourth Creek						
Atmospheric input	160	400	24	3,8	4.1	1.6
Runoff output	4 600	400	8	2.8	4.4	0.8
Third Creek						
Atmospheric input	250	670	19	8.0	3.2	0.5
Runoff output	980	510	8	5.0	3.8	0.9
First Creek						
Atmospheric input	120	430	18	3.5	1.3	0.5
Runoff output	80	30	1	1.2	1.1	0.2
Plantation Hills						
Atmospheric input	60	340	11	3.2	0.8	0.8
Runoff output	20	30	1	0.4	0.4	0.04

lb/acre x 1.12 = kg/ha

Tulsa, Oklahoma--

The Tulsa study was an investigation of storm runoff pollution as it relates to land activity and precipitation. Sampling points were set up for 15 test areas in Tulsa and regression analysis was used to relate pollutant loading to surface characteristics such as area, slope, population density, and land use or to precipitation variables such as intensity, total volume, time from start of storm and time from antecedent event. The pollutant concentrations found in the 15 areas are shown in Table 30 and the relationship between pollutants and significant variables is summarized graphically in Table 31. Some of the basic observations developed from this study include:

- The principal sources of pollutants are washoff from impervious area and erosion of drainage channels.
- Bacterial pollution can be related to the general sanitary condition of the sites.
- Pollutant concentrations decreased with time from the start of the storm and time from the antecedent event. Solids and bacteria increased with intensity of the storm.
- For residential areas, pollution increases with population density and degree of development.

TABLE 30. POLLUTANT CONCENTRATIONS IN STORMWATER RUNOFF, TULSA, OKLAHOMA [14]

	Average value, mg/L					
Site No. and land use	TSS	BOD	COD	Organic nitrogen	OPO_4-P	Fecal coliforms[a]
1. Light industrial	2052	13	110	1.11	1.14	0.94
2. Commercial	169	8	45	0.95	0.28	1.90
3. Residential	280	8	65	1.48	0.62	3.30
4. Industrial-residential	340	14	103	0.97	0.34	0.77
5. Residential	136	18	138	0.72	0.28	1.50
6. Industrial	195	12	90	0.65	0.28	18.00
7. Residential	84	8	48	0.80	0.22	0.12
8. Residential	240	15	115	0.69	0.38	0.45
9. Residential	260	10	117	0.67	0.33	0.29
10. Commercial	300	11	107	0.83	0.23	0.30
11. Residential-commercial	401	14	116	0.66	0.36	0.62
12. Airport	89	8	45	0.39	0.18	0.01
13. Residential	332	15	88	1.46	0.39	0.18
14. Golf course	445	11	53	0.96	0.32	0.37
15. Residential	183	12	42	0.36	0.26	0.35
Total						
Mean, mg/L	367	12	86	0.85	0.38	0.42
Range	0-6378	1-39	14-405	0-5.32	0-4.93	0-470
No. of samples	464	480	425	393	389	358

a. 1000 organisms/100 mL.

TABLE 31. PRECIPITATION AND LAND USE FACTORS AFFECTING POLLUTANT CONCENTRATIONS IN TULSA, OKLAHOMA

	Total solids	Suspended solids	BOD	COD	Organic nitrogen	Ortho-phosphate	Total coliforms	Fecal coliforms
Time from start of storm, h	0	X	●	●	X	X	●	0
Rain, in.	0	X	●	●	X	X	●	0
Intensity, in./h	●	X	●	0	X	X	0	0
Time from antecedent event, h	0	X	0	0	X	X	0	●
Amount of antecedent event, in.	0	X	●	0	X	X	●	0
Intensity of antecedent event, in./h	0	X	0	0	X	X	●	●
Area of basin, acres	●	0	0	0	0	0	0	0
Length of main stream, ft	0	0	●	0	0	0	0	0
Slope of drainage area, %	●	0	0	0	●	●	0	0
Environmental index	0	0	0	0	0	0	●	0
Covered storm sewers, mi	0	0	0	●	0	0	0	0
Arterial streets, %	●	●	●	0	●	0	0	0
Other streets, %	0	0	0	0	●	0	●	0
Residential density, people/acre	0	0	0	●	0	0	●	0
Industrial land, %	●	●	0	0	0	●	0	0
Unused land, %	●	●	0	0	0	●	0	0

Note: 0 no significant correlation
X not investigated
● significant correlation between factor and pollutant

in. x 2.54 = cm
acres x 0.405 = ha
ft x 0.305 = m
mi x 1.61 = km

Oklahoma City, Oklahoma, and Santa Clara County, California--

Data for two regional "Section 208" studies have been gathered. In both cases an effort was made to evaluate runoff from different land use classifications within the study area and compare the data for ultimate use in a planning process. Summaries of the results to date are shown in Table 32. The Oklahoma data clearly show the relatively high potential impact of a central urban core and indicate that pollution concentration increases with increasing population density. The Santa Clara samples also show that some parameters appear to be related to land use.

TABLE 32. POLLUTANT CONCENTRATIONS IN STORMWATER RUNOFF, OKLAHOMA CITY, OKLAHOMA, AND SANTA CLARA COUNTY, CALIFORNIA mg/L

Oklahoma City, Oklahoma

Land classification	TSS	BOD	COD	Kjeldahl nitrogen	Total nitrogen	PO_4-P	OPO_4-P	Lead	Fecal coliforms[a]
Central city	202	32	289	4.2	4.7	1.7	1.7	0.78	24 000
Central city	355	33	125	2.3	3.5	2.1	2.1	0.44	130 000
Residential urban	88	30	101	2.3	4.0	0.66	0.70	0	14 000
Suburban	6	9	22	0.5	2.0	0.06	0.06	0	22 000
Rural	87	5	45	1.1	1.9	0.49	0.46	0	10 000
Total									
Mean	147	22	116	2.1	3.2	1.00	1.00	0.24	40 000
Range	3-639	1-34	0-392	0-0.5	0.4-15.8	0-3.69	0-3.69	0-2.23	8 000-160 000
No. of samples	40	40	40	40	40	39	40	12	20
Woodland	63	7	44	0.6	0.6	0.05	......	0	2 900

Santa Clara County

Land classification	TSS	VSS	BOD	COD	Nitrogen	PO_4-P	Lead
Residential	457	107	37	...	7.2	0.34	....
Residential	512	92	17	152	2.9	0.18	0.4
Commercial	120	53	23	255	9.4	0.29	0.98
Industrial	123	52	14	119	6.0	0.14	(5.0)[b]
Industrial	42	21	11	104	3.8	0.16	0.65
Mixed	341	78	24	......	5.9	0.24	....
Total							
Mean	284	70	20	147	5.8	0.23	0.75
Range	0-2 252	0-604	2-73	81-255	1.0-26.9	0.05-.90	0.35-1.45
No. of samples	78	77	64	5	73	66	8

a. Organisms/100 mL.

b. Includes industrial discharge--not used for mean.

Pullach, Germany--

The one non-American study cited was an extensive sampling effort in southern Germany where approximately 1200 samples were taken for 62 runoff events. The program covered a complete year in a 23 ha (57 acre) suburban basin that was described as having "the character of a small town" with residential areas and a town center. The flow weighted averages from the sampling program were shown in Table 21. The length of the program allowed the investigation of pollutant concentration variation with seasons. The climate of Pullach is similar to the northeast and midwest sections of the United States with 95 cm (37 in.) of rain, 108 cm (42 in.) of snow, and mean monthly temperatures ranging from -2°C to 17.5°C (28°F to 64°F). The variation for key pollutants is shown in Figure 14. The high winter loading of suspended solids is attributed to washoff of deicing grit and roadway material worn away by studded snow tires. The peak month was April when the accumulated winter solids would be removed by spring rains. The BOD concentrations did not follow any explainable pattern. The bacterial fluctuations appear to be based on climatic conditions with higher concentrations in the warm summer months that would allow for longer survival times.

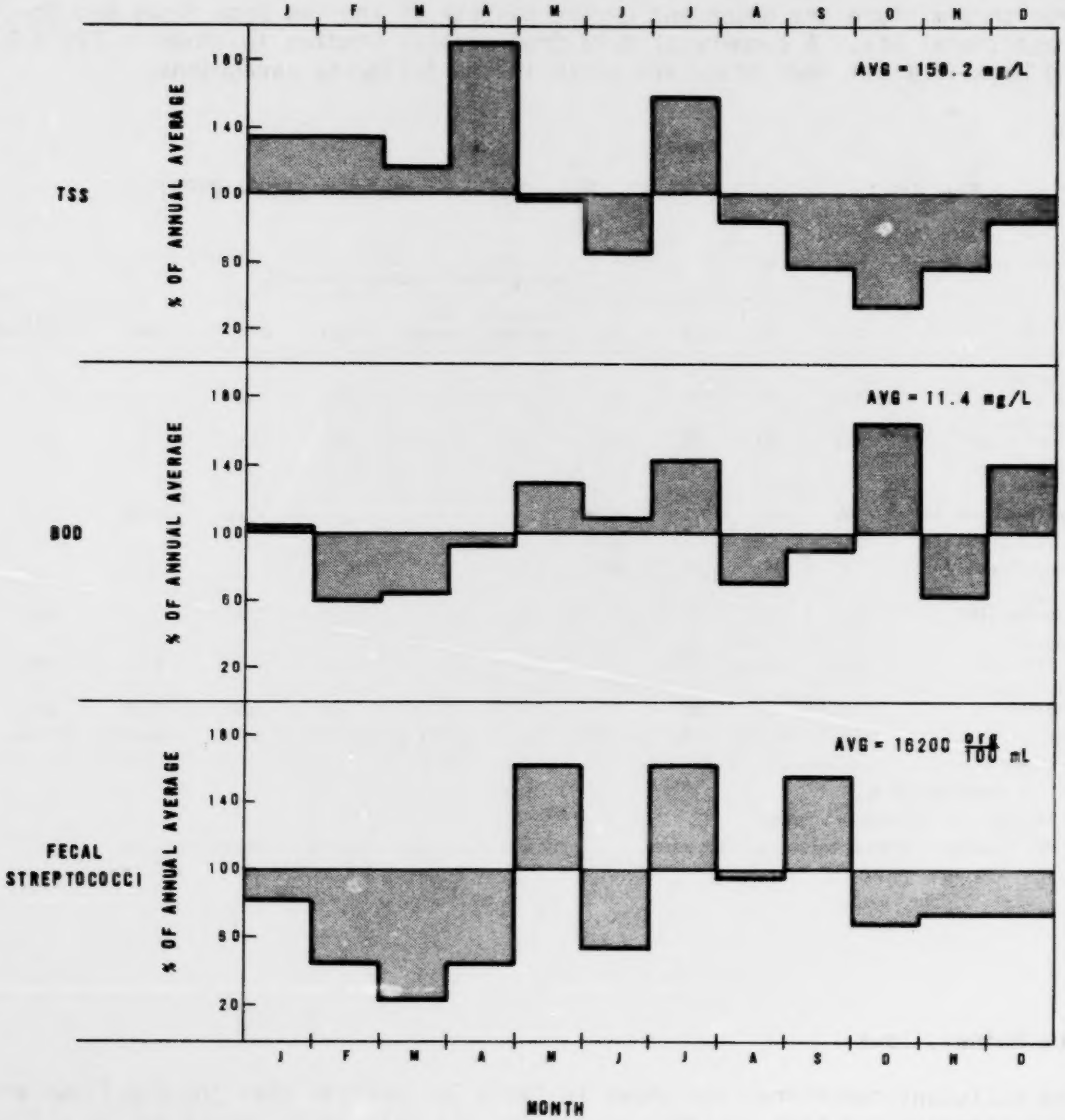


Figure 14. Variation of pollutant concentrations by month in Pullach, Germany [3].

Combined Sewer Overflows

In many cities, especially those with older sewer systems, the storm runoff and sanitary sewage flow in the same conduits and overflow as a mixture when the pipe capacity is exceeded during a storm. Sampling programs have been developed to characterize the quality of the overflows for the predesign of abatement programs. The pollutant values are a combination of runoff pollutant concentrations, as described in the previous section, and sanitary sewage pollutant concentrations. Site specific concentrations that result from this mixture are dependent on the quality of the two base flows and the proportional mix. A summary of data from several studies is shown in Table 33 and highlights of each study are given in the following paragraphs.

TABLE 33. POLLUTANT CONCENTRATIONS IN COMBINED SEWER OVERFLOWS

	Average pollutant concentration, mg/L									
	TSS	VSS	BOD	COD	Kjeldahl nitrogen	Total nitrogen	PO_4-P	OPO_4-P	Lead	Fecal coliforms[a]
Des Moines, Iowa [11]	413	117	64	...	...	4.3	1.86	1.31	....	....
Milwaukee, Wisconsin [15]	321	109	59	264	4.9	6.3	1.23	0.86	....	....
New York City, New York										
Newtown Creek [16]	306	182	222	481	...	....	.....	....	0.60	....
Spring Creek [17]	347	...	111	358	...	16.6	4.5[b]	....	....	....
Poissy, France [18][c]	751	387	279	1005	...	43	17[b]	....	....	....
Racine, Wisconsin [19]	551	154	158	...	...	....	2.78	0.92	....	201
Rochester, New York [20]	273	...	65	...	2.6	...	.....	0.88	0.14	1140
Average (not weighted)	370	140	115	367	3.8	9.1	1.95	1.00	0.37	670
Range	273-551	109-182	59-222	264-481	2.6-4.9	4.3-16.6	1.23-2.78	0.86-1.31	0.14-0.60	201-1140

a. 1000 organisms/100 mL.

b. Total P (not included in average).

c. Not included in average because of high strength of municipal sewage when compared to the United States.

Des Moines, Iowa--

The pollutant concentrations shown in Table 34 indicate that the overflows are less concentrated than sanitary sewage for all pollutants except solids [11]. Although the areas sampled had varying percentages of combined and separate sewers, the pollutant concentrations did not appear to be related to the variation.

TABLE 34. POLLUTANT CONCENTRATIONS IN COMBINED SEWER OVERFLOWS, DES MOINES, IOWA [11]

Site	TSS	VSS	BOD	NH_3-N	Total nitrogen	PO_4-P	OPO_4-P	Comments
	Mean value for pollutants, mg/L							
0-2	495	95	44	1.21	2.12	0.72	0.41	6% combined sewers
0-3	144	77	69	4.53	4.87	3.83	2.70	6% combined sewers
0-6	592	181	95	9.42	10.19	3.23	3.02	25% combined sewers
0-7	195	62	50	1.84	3.0	2.32	0.99	33% combined sewers
0-8	410	142	68	3.22	4.32	2.27	1.66	69% combined sewers
0-8a	303	101	77	4.94	5.54	...	1.98	81% combined sewers
CSO summary								
Mean, mg/L	413	117	64	3.36	4.32	1.86	1.31	
Range, mg/L	10-1 549	2-582	9-220	0-27.4	0.2-27.6	0.27-6.6	0.01-84	
No. of samples	64	64	69	56	56	39	47	
Sanitary sewage	230	170	195	24.3	25.1	5.7	3.6	

Milwaukee, Wisconsin--

The data presented in Table 35 summarize the influent flow quality to a screening pilot plant operated at the Hawley Road combined sewer overflow; data from four test periods and a breakdown between the first-flush and the remainder of the storm for a fifth test period are shown [15]. It is generally assumed that quantities of sanitary sewage settle out in the large combined sewers during periods of low flow and, as a storm event begins, the material is resuspended. This means that the first portion of a storm will carry exceptionally high pollutant concentrations. The data confirm that the initial part of an overflow at Hawley Road has pollutant concentrations that are much higher than later parts of a storm. The concentration of the first-flush appears to be related to the number of dry days preceding the storm event. Solids accumulate over the dry period and longer intervals provide a larger mass of pollutants for resuspension. The relationship between dry intervals and first-flush concentrations is shown in Figure 15. Although a simple equation would not provide a good correlation for the points, it is apparent that the dry period influences pollutant concentration.

TABLE 35. POLLUTANT CONCENTRATIONS IN COMBINED SEWER OVERFLOWS, MILWAUKEE, WISCONSIN (HAWLEY ROAD) [15]

	Pollutant concentrations, mg/L							
Sampling period	TSS	VSS	BOD	COD	Kjedahl nitrogen	Total nitrogen	PO_4P	OPO_4-P
Preliminary data								
1967-1968	400	113	49	336	....	....	.....	.....
1971	435	146	64	209	....	6.3	.....	0.86
1973	129	52	42	...	4.9	....	0.99	.....
1974	162	87	74	...	....	....	1.47	.....
Summary								
Mean	321	109	59	264	4.9	6.3	1.23	
Range	32-2 158	12-720	4-318	26-1 410	1.9-14.3	1.0-27.9	0.25-4.04	0.06-0.93
No. of samples[a]	55	42	49	37	10	21	20	21
First flush								
1969-1970	522	308	186	581	....	17.6	2.7	......
Remainder of storm								
1969-1970	166	90	49	161	....	5.5	.....	......

a. Samples reported as averaged for overflow events.

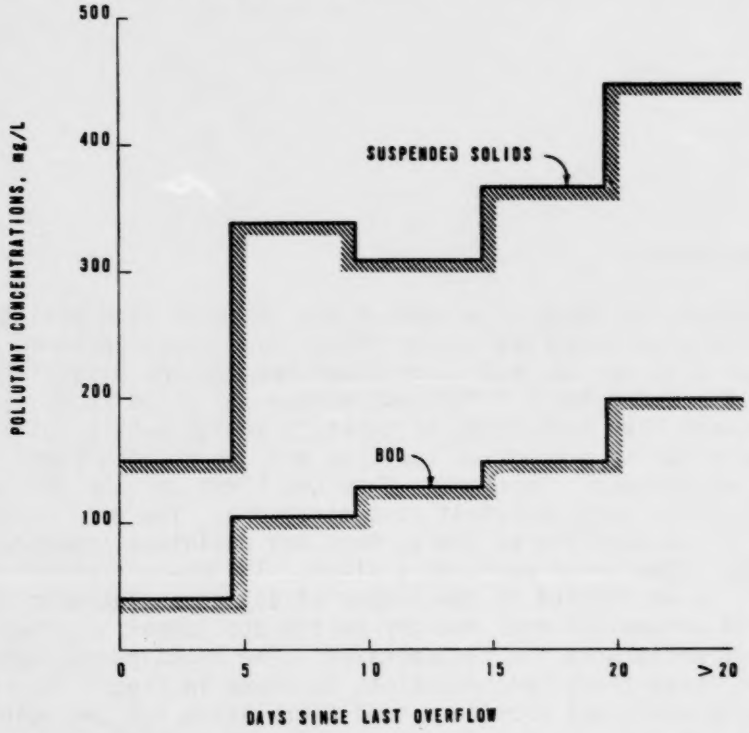


Figure 15. Average pollutant concentration versus preceding dry-weather period, Milwaukee, Wisconsin [15].

In a second study at Milwaukee's Humboldt Avenue Project [21] the first-flush phenomenon was also noted. Values for suspended solids and BOD during progressive overflow time intervals are shown in Figure 16. The data shown consist of average concentrations for samples from 97 storm events.

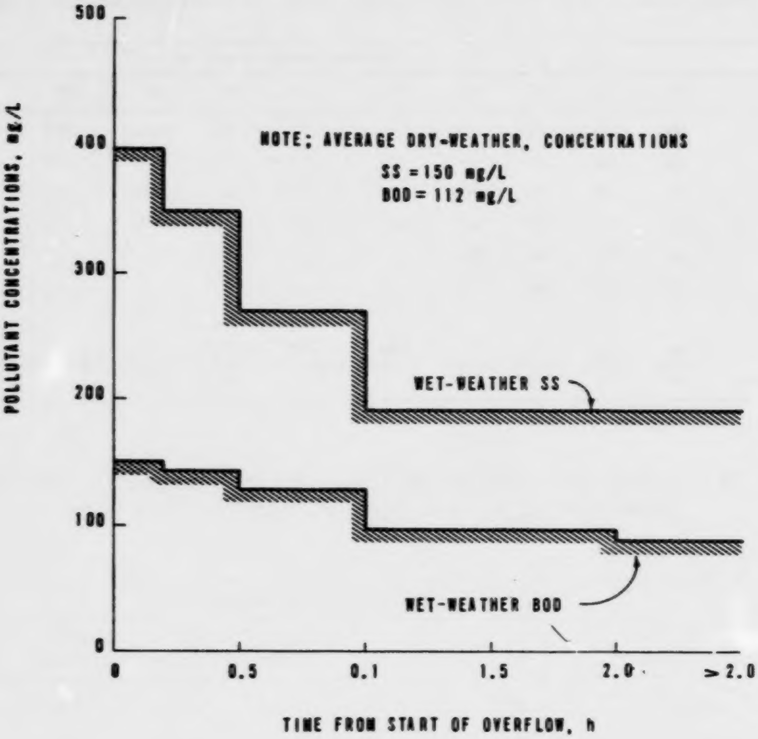


Figure 16. Overflow quality versus time at Milwaukee, Wisconsin [21].

New York City--

Combined sewer overflow sample data are available for two sites in New York City--the Newtown Creek and Spring Creek Water Pollution Control Facilities. At Newtown Creek an ultra high rate filtration pilot study was performed in 1976-1977 to test the feasibility of treating combined sewer overflows. The data in Table 36 are for composite samples from the pilot plant influent during six storms.

The second set of data from New York City is the result of a study of the ecosystem and sources of pollution for Jamaica Bay. The objective of the project was to evaluate an ongoing combined sewer overflow control program by developing estimates of pollutant input to the bay and modeling receiving water quality to determine the impacts of alternative control measures. Characerization of combined sewer overflows was accomplished by sampling five basins during a period from March 1969 until January 1971. The data are summarized by drainage basin in Table 37. Generally, the solids concentrations are much higher than sanitary sewage while the organics are similar to sewage.

TABLE 36. POLLUTANT CONCENTRATIONS IN COMBINED SEWER OVERFLOWS, NEW YORK CITY, NEW YORK - NEWTOWN CREEK [16]

	Pollutant concentrations, mg/L										
	TSS	VSS	BOD	COD	Pb	Cd	Cr	Cu	Hg	Ni	Zn
Storm 0	608	...	315	562	1.28	.035	6.0	0.6	.0022	0.80	1.00
Storm 1-1	232	180	188	498	0.33	.035	0.55	0.32	.0008	0.28	0.57
Storm 1-2	132	120	130	344	0.19	.0087	0.35	0.26	.0002	0.22	0.46
Storm 2	248	184	240	483	0.6	.018	0.90	0.44	.0002	0.50	0.50
Storm 3	380	252	235	551	....	......	....	....	......	....	....
Storm 4	236	176	222	449	....	......	....	....	......	....	....
Summary											
Mean	306	182	222	481	0.60	.024	1.95	0.41	.0009	0.27	0.63
Range	132-608	120-252	130-315	344-562	0.19-1.28	.0087-0.35	.35-6.0	.26-0.6	.0002-.0022	.08-.50	.46-1.00
No. of samples[a]	6	5	6	6	4	4	4	4	4	4	4

a. Composite samples taken over storm duration.

TABLE 37. POLLUTANT CONCENTRATIONS IN COMBINED SEWER OVERFLOWS, NEW YORK CITY, NEW YORK (SPRING CREEK) [17]

	Pollutant concentrations, mg/L					
Sites	TSS	BOD	COD	Total nitrogen	Total phosphorus	Soluble phosphorus
Paerdegat	81	79	...	15.9	11.7	0.9
Hendrix	255	127	383	17.3	5.3	3.9
Spring Creek West	341	95	259	20.6	3.8	2.8
Spring Creek East	556	142	424	18.2	4.4	3.3
Thurston	223	89	382	10.5	2.4	1.8
Summary						
Mean	347	111	358	16.6	4.5	2.8
Range[a]	51-1 050	7-340	54-600	1.9-38	....	0.17-10
No. of storms	41	31	10	25	29	29
Sanitary sewage	145	119	391	44	9	6

a. Data reported as flow weighted mean for a storm.

Poissy, France--

Eight storm events were sampled during a September-to-July period in Poissy, France. The samples were part of a characterization study to develop annual loads from storm events in the Paris region. The results are listed in

Table 38 and compared to the sanitary sewage concentrations for the same system. It is apparent that the sewage is more concentrated than an average American sewage.

TABLE 38. POLLUTANT CONCENTRATIONS IN COMBINED SEWER OVERFLOWS, POISSY, FRANCE [18]

	Pollutant concentrations, mg/L					
	TSS	VSS	BOD	COD	Total nitrogen	Total phosphorus
Storm 1	417	314	147	783	....	....
Storm 3	420	217	130	558	18.5	6.3
Storm 4	307	176	80	267	13.1	3.5
Storm 6	275	161	298	1 463	61	13
Storm 7	1 545	636	424	1 742	65	33
Storm 8	845	321	250	464	33	25
Storm 9	976	453	277	861	38	9.4
Storm 10	1 223	821	628	1 903	74	27
CSO totals						
Mean	751	387	279	1 005	43	17
Range	275-1 223	161-821	80-628	267-1 903	13.1-74	3.5-27
No. of samples	8	8	8	8	7	7
Sanitary sewage	280	230	330	960	73	26

From these sampling efforts, annual loads of pollutants were estimated for the basin. Three primary pollutants were estimated with suspended solids as a function of runoff flow rate, COD as a function of runoff volume, and BOD as a function of volume or runoff rate. The mass loadings were estimated for each storm and summed over a year to estimate annual loads.

Racine, Wisconsin--

The Racine project was an evaluation of three full-scale demonstration systems for the treatment of combined sewer overflows. Three sets of raw overflow data were available from the preliminary investigation prior to setup and two additional sets were compiled from the demonstration runs. The concentrations are tabulated in Table 39.

There are some large differences in pollutant concentrations for different sites and time periods. The temporal variations were attributed to sampling methods and changes in the sewer system; the variation between sites was attributed to the fact that sewers tributary to Site 1 flow near capacity and will prevent deposition during dry weather, while sewers tributary to Site 2 flow well below capacity. The dry-weather deposition in the sewers will be exhibited as a "first flush" of heavily concentrated overflow. During the 1971 preliminary phase, individual samples were taken during the 10 events monitored at Site 2. Average values were computed for time periods from the start of overflow and plotted in Figure 17. The graph shows the tendency for high initial concentrations.

TABLE 39. POLLUTANT CONCENTRATIONS IN COMBINED SEWER OVERFLOWS, RACINE, WISCONSIN [19]

	Pollutant concentrations, mg/L						
	TSS	VSS	BOD	TOC	PO_4-P	OPO_4-P	Fecal coliform[a]
Site 1 - overflow 1, 1971	298	...	79	98	.....	0.64	10 300
Site 1 - overflow 3, 1971	669	...	212	194	.....	2.07	21 900
Site 2 - overflows 7-8, 1971	669	...	212	238	.....	0.75	10 100
Site 1, 1974	266	134	93	95	3.13	.....	609 000
Site 2, 1974	661	178	110	122	2.38	.....	416 000
Summary[b]							
Mean	551	154	158	172	2.78	0.92	201 000
Range	38-2 070	26-475	27-510	24-459	1.0-5.62	0-3.17	540-4 400 000
No. of storms	108	41	108	106	41	69	107

a. Organisms/100 mL.

b. Based on storm event composites.

Rochester, New York--

The data base for Rochester was gathered as part of a study to optimize storage treatment capacities in the combined sewer system. A large number of samples were taken at 12 overflow sites during an 18 month period. The results are shown in Table 40 by overflow site along with an indication of principal land uses for areas that are tributary to each overflow. Concentrations are not readily correlated to land use. The "first flush" phenomenon occurred in Rochester as it did in Milwaukee and Racine. Average pollutant concentrations for storm intervals are graphed in Figure 18 and show a significantly decreasing value for increasing time.

Summary of Discharge Data

The average pollutant concentrations for urban runoff and combined sewer overflows are compared to background pollution and sanitary sewage in Table 41. The background data are the reported range of quality constituents from the USGS National Hydrologic Benchmark Network that was established to obtain a natural background. The ranges are for average values across the country. The sanitary sewage values represent common design values used to characterize a medium strength municipal sewage.

The ten common pollutants listed in Table 41 have been extensively surveyed in stormwater studies as indications of pollution from solids, organics,

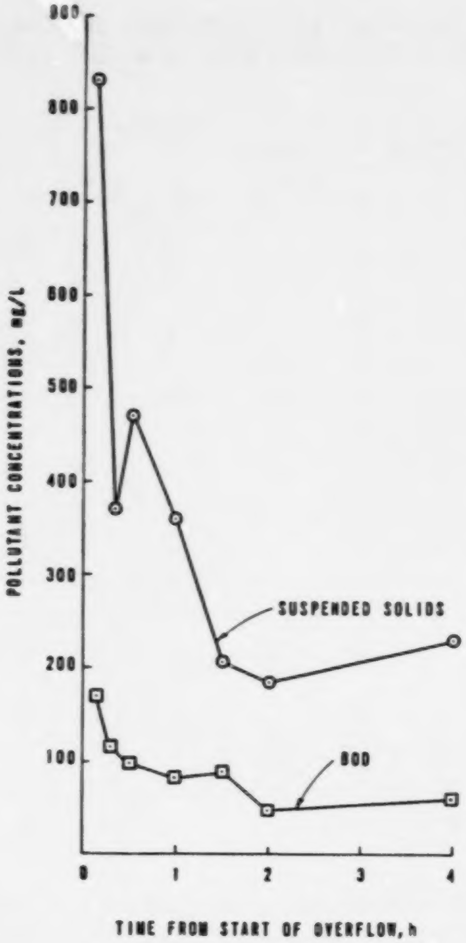


Figure 17. Overflow quality versus time at Racine, Wisconsin [19].

nutrients, toxic metals, and pathogens. In most cases these parameters will be sufficient to characterize runoff problems and impacts. However, in recent years there has been an increasing awareness of potential danger to receiving waters from low concentrations of metals, pesticides, and microorganisms. Typical values obtained for parameters in each category are shown in Tables 42, 43, and 44. The values were obtained for a variety of reasons under different conditions and are presented as representative of ranges that may be expected

Urban rainfall-runoff-quality data have been assembled on magnetic tape by the University of Florida [22]. So far eight sites are incorporated in the project including Broward County, Florida; San Francisco, California; Racine, Wisconsin; Lincoln, Nebraska; Windsor, Ontario; Durham, North Carolina;

TABLE 40. POLLUTANT CONCENTRATIONS IN COMBINED SEWER OVERFLOWS, ROCHESTER, NEW YORK [20]

	Mean pollutant concentration, mg/L						
Site	TSS	BOD	TOC	Kjeldahl nitrogen	TIP[a]	Fecal coliform[b]	Principal land uses
7	289	...	53	5.5	0.69	1.12	Residential
8	203	75	44	4.3	0.48	0.58	Commercial-Residential
9	219	129	70	11.0	1.00	0.53	Residential-Industrial
16	368	99	79	2.8	0.32	0.85	Residential-Commercial
17	142	34	30	2.2	2.56	1.10	Residential
18	192	66	38	3.7	0.77	1.11	Residential
21	124	71	50	7.5	0.71	1.69	Residential
22	512	...	223	7.0	1.97	1.90	Residential
25	207	52	35	3.8	0.42	0.81	Commercial-Residential
26	395	23	28	0.9	0.14	0.12	Commercial-Residential
28	244	63	48	2.4	0.33	1.18	Residential
31	173	79	49	2.5	1.18	1.22	Residential-Commercial-Industrial
Summary							
Mean	273	65	72	2.6	0.88	1.14	
Range	4-29 590	0-610	0-1 420	0-94	0-77.6	0-70	
No. of samples	1 976	1 184	2 358	2 383	2 385	1 709	

a. Total inorganic phosphate as P.

b. Million organisms/100 mL.

Lancaster, Pennsylvania; and Seattle, Washington. The computerized data will be available for characterization studies and the calibration and verification of runoff models. There are many additional past and current studies that can be added to the project to expand the data base.

Normalization of Data

The discharge data in the preceding subsection have been developed as concentrations for the two basic categories of storm discharges--urban stormwater runoff and combined sewer overflows. It is evident from the comparison in Table 41 that normalization by discharge system type is a primary consideration. The values in this table represent a random cross-section of sampling experience for the two types of systems and as such are a valid starting point for the analysis of urban stormwater discharges. The brief descriptions of the studies behind the data indicate that the samples represent mixed urban areas for extended time periods. The values may not be representative for small homogeneous drainage basins or individual storm events.

In most cases, an investigator is interested in the mass of pollutants that are tributary to the receiving water. The concentrations developed must be

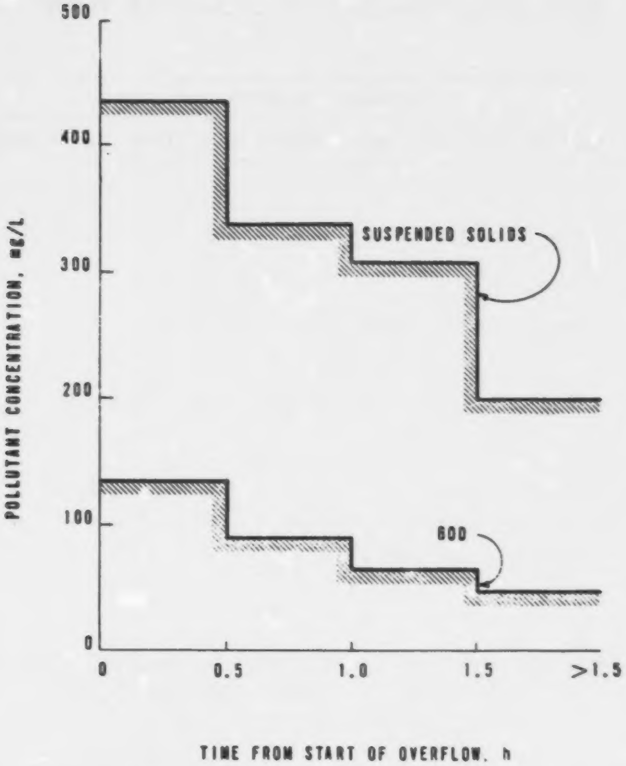


Figure 18. Overflow quality versus time at Rochester, New York [20].

TABLE 41. COMPARISON OF TYPICAL VALUES FOR STORMWATER DISCHARGES[a]

	TSS	VSS	BOD	COD	Kjeldahl nitrogen	Total nitrogen	Total PO_4-P	OPO_4-P	Lead	Fecal coliforms
Background levels [9]	5-100	...	0.5-3	20	.....	0.05-0.5[b]	0.01-0.2[c]	...	<0.1	....
Stormwater runoff	415	90	20	115	1.4	3.10	0.6	0.4	0.35	13 500
Combined sewer overflow	370	140	115	367	3.8	9.10	1.9	1.0	0.37	670 000
Sanitary sewage [23]	200	150	200	500	40	40	10	7	....	....

a. All values mg/L except fecal coliforms which are organisms/100 mL.

b. NO_3 as N.

c. Total phosphorus as P.

TABLE 42. METAL CONCENTRATIONS IN STORMWATER RUNOFF AND OVERFLOWS

Site	Pollutant concentrations, mg/L								
	Cadmium	Chromium	Copper	Nickel	Zinc	Iron	Lead	Manganese	Magnesium
New York City, New York [24]	0.025	0.16	0.46	0.15	1.6	.....	....	....	..
Durham, North Carolina [12]	......	0.23	0.15	0.15	0.36	12.	0.46	0.67	10
Rochester, New York (2 sites) [20]	0.0021	0.0065	0.086	0.013	0.24	1.66	0.14	....	..
Drinking Water Standards [6]	0.01	0.05	1.0	.....	5.0	0.3	0.05	0.05	..

a. Maximum permissable concentrations.

TABLE 43. PESTICIDE AND HERBICIDE CONCENTRATIONS IN STORMWATER RUNOFF AND OVERFLOWS
Parts per Trillion

Pesticide and herbicide	1971 Site 11	Racine, Wisconsin [19]					Hayward California [25] 1971-1972		Drinking water standards [26][a]
		1973		1974					
		Site 11A	Site 11	Site 1	Site 11	Site 11A	Average	Maximum	
Lindane	<1	90	130	<1	<1	<1	31	150	5 000
Heptachlor	<1	<10	<10	<1	<1	<1	0	0	100
Aldrin	14	<10	<10	<1	<1	<1	4	70	1 000
Heptachlor epoxide	16	<10	<10	32	23	<1	0	0	100
Methozchlor	58	<15	<15	<1	<1	<1	...	.....	10^6
Dieldrin	<1	120	<10	<1	14	<1	90	190	1 000
Endrin	<1	260	100	<1	<1	<1	0	0	500
Methyl parathion	..	...	...	..	..	..	0	0	......
Parathion	..	...	...	..	..	..	0	0	......
DDT	..	...	...	61	89	5	130	630	50 000
DDD	..	...	...	26	34	5	6	80	......
DDE	..	...	...	<1	<1	<1	16	100	......
Chlordane	..	...	...	..	..	..	560	2 400	3 000
Diazinon	..	...	...	..	..	..	195	260	......
Malathion	..	...	...	..	..	..	128	540	......
Silvex	..	...	...	..	..	..	81	560	30 000
2,4-D	..	...	...	..	..	..	570	6 400	20 000
2,4,5-T	..	...	...	..	..	..	63	200	2 000

a. Maximum permissible concentrations.

TABLE 44. MICROORGANISMS IN STORMWATER RUNOFF AND OVERFLOWS

	Organisms/100 mL							Enterovirus, PFU/10 L
	Total coliforms	Fecal coliforms	Fecal streptococci	Enterococci	*Staph. aureus*	*P. aeruginosa*	*Salmonella* sp.	
Baltimore, Maryland [26]								
Stormwater	120 000	24 000	170 000	50 000	38	1 100	0.13	34
Combined sewer overflow	590 000	230 000	260 000	72 000	38	5 900	0.59	92
Houston, Texas [27]								
The Woodlands, a developing community								
Upstream	258 000	1 300	650	......	450	85	<38	..
Downstream	403 000	1 800	2 020	......	2 240	260	<62	..
Westberry Square, a residential area	30 100 000	22 000	13 100	......	8 120	7 560	<33	..

paired with volumes of runoff or overflow to get a loading. Consequently, the concentrations used should have been developed as flow weighted values so that the combination of concentration and flow gives an accurate loading. Most of the concentrations developed in the literature are simple averages of random samples and give undue weight to low volume-high concentration flow. A flow weighted concentration can also be expressed in different dimensional terms. Mass of pollutant per unit area of basin per unit depth of runoff (kg/ha per cm) has been suggested as a replacement for mg/L and parts per million [28].

Normalization can be extended to other factors that influence pollutant concentrations. The three most common categories are land use classes, runoff volume or rate, and time from start of event.

Land Use--

Normalization of data as a function of land use is important to areawide characterization studies because of the need to rank the pollutant potential of different areas and the need to project pollution loadings for future growth options. Ranking allows the concentration of available money on problem areas to maximize the decrease in pollution per dollar spent and also establishes a priority list for phased development of control programs. Projection of future pollution potential by land use allows planners to direct potentially damaging growth away from environmentally sensitive areas.

The effect of land use on solids deposition is shown in Section 4. The data for that evaluation were obtained by sweeping street surfaces in areas that could easily be identified as residential, commercial, or industrial. Studies of stormwater discharges and overflows have not been able to concentrate on small areas with easily definable land use characteristics. Most samples have been taken from mixed basins with at least two classifications and some open space.

An analysis of quality based on land use was developed in an EPA report [29]. The basis of the analysis was a tabulation of BOD concentration in storm discharges for residential areas from several cities. The remaining parameters and land use classifications were developed as ratios of this discharged BOD, using data acquired from the street surface sampling projects mentioned previously. The methodology was originally developed to project annual loads. The factors were converted to concentrations in another EPA report by assuming runoff coefficients (runoff/precipitation) for each land use area [30]. The residential data must also be modified by a population factor $p_1(PD_d)$ which is a function of population density.

$$p_1(PD_d) = 0.142 + 0.134\,(PD_d)^{0.54} \quad \text{(SI units)} \qquad (5\text{-}3a)$$

$$p_1(PD_d) = 0.142 + 0.218\,(PD_d)^{0.54} \quad \text{(U.S. customary units)} \qquad (5\text{-}3b)$$

where PD_d = developed area population density, people/ha (people/acre)

The factor is equal to one at a density of 31 people per hectare. The concentrations and runoff coefficients are presented as a function of land use in Table 45.

TABLE 45. POLLUTANT CONCENTRATIONS AS A FUNCTION OF LAND USE

	Pollutant concentrations, mg/L					
	TSS	VSS	BOD	PO_4-P	Total nitrogen	Runoff coefficient
Urban runoff						
Residential[a]	240	140	12	0.16	1.9	0.3
Commercial	140	90	20	0.16	1.9	0.7
Industrial	215	105	9	0.17	2.0	0.6
Other developed areas	17	16	1	0.02	0.4	0.1
Combined sewer overflows						
Residential	990	570	50	0.67	8.0	0.3
Commercial	580	360	85	0.64	7.7	0.7
Industrial	880	430	35	0.66	8.4	0.6
Other developed areas	70	70	3	0.08	1.6	0.1

a. Modify residential values by factor $p_1(PD_d) = 0.142 + 0.134\,(PD_d)^{0.54}$. (PD_d) = people/ha.

Because of the methodology used to develop this table, the values for combined sewage overflows are very questionable. The BOD values in combined sewage are principally attributed to the sanitary sewage and the ratios of BOD to other pollutants are much different in sanitary sewage as compared to street solids. The solids values estimated for combined sewage are high and the nutrient values are low. Satisfactory values for combined sewage have not yet been developed. Data could be developed at a specific site by combining the urban runoff concentrations in Table 45 with local estimates of the concentrations found in domestic or industrial sewage. The result would have to be volume weighted for the amounts of sewage and runoff expected at the overflow.

Precipitation and Runoff Characteristics--

The main goal of normalization of quality data as a function of precipitation or runoff is to make the loadings correlate to shorter time periods and storm events. Precipitation data are generally available in sufficient historical quantity to permit the estimation of probability of occurrence of rainfall volume, duration, intensity, and intervals. Several methodologies are also available for estimating runoff characteristics based on precipitation. All that is required to estimate pollutant loadings is to correlate runoff pollution concentrations to the runoff characteristics.

The work at Durham, North Carolina, was discussed previously and the equations developed there were presented in Table 27 [12]. The concentration of pollutants was developed as a function of runoff rate and time from the start of the storm in the form:

$$\text{concentration} = A(\text{rate of runoff})^{x}\ (\text{time})^{y} \qquad (5\text{-}4)$$

This equation will predict the pollutant concentrations at any time during a runoff event and can be integrated to get mass loading for an event. Samples from Durham indicated a very high solids content in the runoff and this probably means erosion problems in the natural drainage channels. Rate of runoff may not be an important parameter for areas with buried storm sewers or lined channels.

Regression analysis of pollutant concentrations as functions of precipitation was also attempted at Rochester, New York, and Tulsa, Oklahoma. The equations are presented in Table 46. Rochester has combined sewers and Tulsa has separate storm channels.

The relation between runoff characteristics and pollutant concentrations were shown in graphs developed for Des Moines, Iowa [11] and reproduced as Figure 19. The plots show the mass of pollutant from a unit area as a function of runoff. The values were based on composite samples for storm events and are valid only for complete events. In these graphs a straight linear curve would indicate a constant value for concentration regardless of the runoff volume. Most of the curves are slightly concave indicating a small decrease in composite concentration as the total volume of runoff increases.

This same relation was developed by regression analysis for Creteil, France, during a study of the effects of stormwater on Creteil's Lake [31]. The equations relate BOD and suspended solids mass loading to the volume of runoff per event. The original equations were:

$$\log(SS) = 1.298 \log(V) - 1.208 \qquad \text{(SI units)} \qquad (5\text{-}5a)$$

$$\log(SS) = 1.298 \log(V) - 2.85 \qquad \text{(U.S. customary units)} \qquad (5\text{-}5b)$$

$$\log(BOD_w) = 0.545 \log\left(\frac{V}{114}\right) + \log T \quad \text{(SI units)} \qquad (5\text{-}6a)$$

$$\log(BOD_w) = 0.545 \log(V) + \log T - 1.62 \quad \text{(U.S. customary units)} \qquad (5\text{-}6b)$$

$$\log(BOD_e) = 0.82 \log\left(\frac{V}{114}\right) + 0.398 \quad \text{(SI units)} \qquad (5\text{-}7a)$$

$$\log(BOD_e) = 0.82 \log(V) - 2.215 \quad \text{(U.S. customary units)} \qquad (5\text{-}7b)$$

where SS = mass loading, kg (lb)
BOD_w = mass loading due to washoff, kg (lb)
BOD_e = mass loading due to erosion, kg (lb)
V = runoff volume, m^3 (ft^3)
T = time since last rain, d

The equations can be factored for the 45.4 hectare basin and reduced to the forms:

$$SS = 76.2\ R^{1.298} \quad \text{(SI units)} \qquad (5\text{-}8a)$$

$$SS = 228\ R^{1.298} \quad \text{(U.S. customary units)} \qquad (5\text{-}8b)$$

$$BOD = 1.13\ R^{0.82} + T \times 7.48\ R^{0.545} \quad \text{(SI units)} \qquad (5\text{-}9a)$$

$$BOD = 2.16\ R^{0.82} + T \times 1.48\ R^{0.545} \quad \text{(U.S. customary units)} \qquad (5\text{-}9b)$$

where SS and BOD = mass loadings, kg/ha (lb/acre)
R = runoff, cm (in.)
T = time since last rain, d

A comparison of the Des Moines and Creteil data is made in Table 47. The higher values at Creteil may be due to the much higher population density of 220 people/ha (89 people/acre) as compared to approximately 25 people/ha (10 people/acre) in Des Moines.

Time--

Perhaps the most significant quality normalization criterion other than the type of system is related to the time of sampling with respect to the start of the event and the interval between events. Whether the primary cause is first flush or the declining availability of source contaminants, overflow or runoff quality tends to improve in the latter stages of a storm and in the latter storms of a storm series. Normalized comparisons of data from several cities with respect to time are summarized in Table 48. This quality-time relationship is particularly significant in optimizing storage-treatment operations where bypasses or multilevel treatment must be considered.

Summary--

Much more work is needed to develop valid normalization techniques that will simplify the analysis of stormwater problems. The present detailed techniques have not been calibrated at enough sites to test their applicability.

TABLE 46. POLLUTANT CONCENTRATIONS AS A FUNCTION OF PRECIPITATION CHARACTERISTICS

Equation	Correlation
Rochester [20]	
$\mathrm{COD} = 50.17\, X_1^{0.0705}\, X_2^{0.0761}\, X_3^{-0.407}$	0.257
$\mathrm{TSS} = 159.62\, X_1^{0.220}\, X_2^{-0.345}\, X_3^{-0.329}$	0.181
where COD and TSS are in mg/L X_1 = days since last rain X_2 = duration of rainfall, h X_3 = average intensity of rainfall, in./h	
Tulsa [14]	
$\mathrm{In\ (BOD)} = 2.7531 + 0.0086(Z_1) - 0.6484(Z_2) - 0.3674(Z_5)$	0.274
$\mathrm{In\ (COD)} = 4.5757 - 0.0246(Z_1) - 0.2001(Z_2) - 0.0900(Z_3)$	0.215
$\mathrm{In\ (TSS)} = 5.7304 - 0.0144(Z_1) + 0.0572(Z_2) + 0.3004(Z_3)$	0.103
where BOD, COD, TSS are in mg/L Z_1 = time since start, h Z_2 = antecedent amount, in. Z_3 = antecedent average intensity, in./h Z_5 = amount of antecedent event, in.	

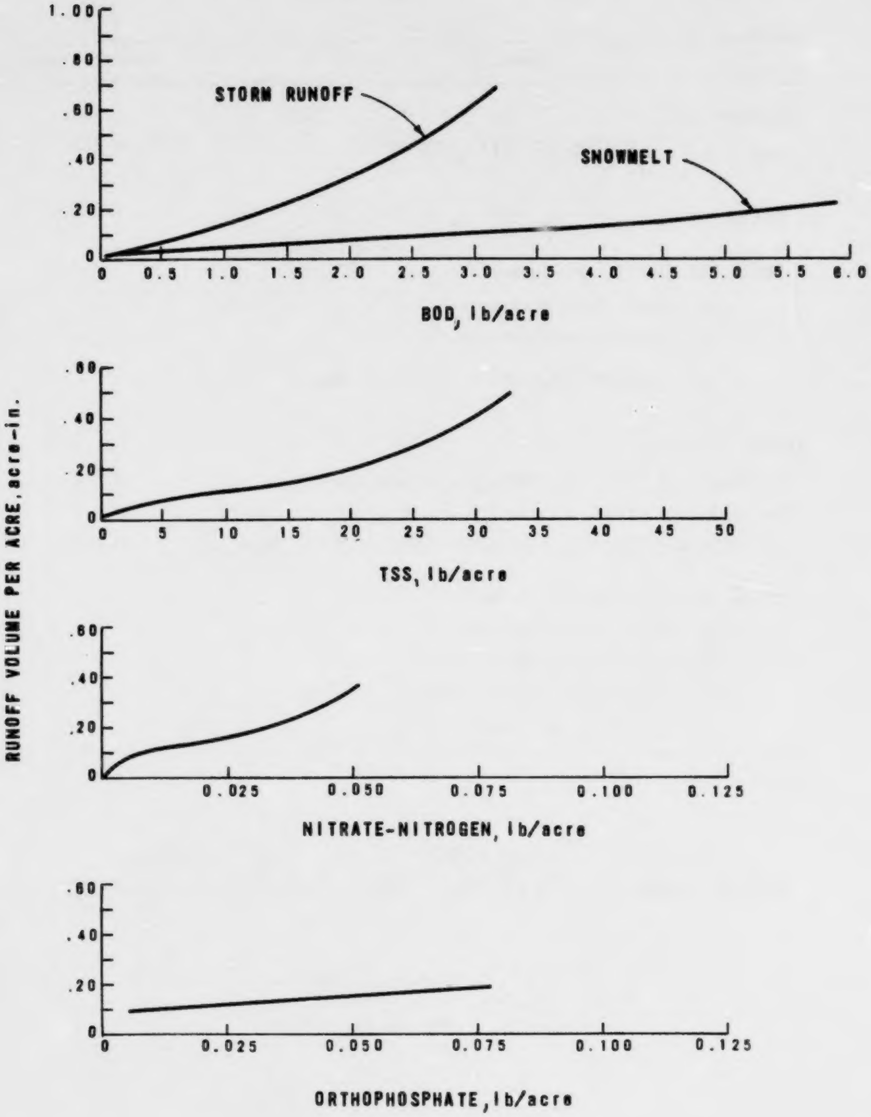
in./h x 2.54 = cm/h

TABLE 47. COMPARISON OF POLLUTANT LOADING ASSUMPTIONS AT DES MOINES, IOWA, AND CRETEIL, FRANCE

	Runoff, in.	Pollutant, lb/acre	
		Des Moines	Creteil
BOD[a]	0.1	0.75	2.4
	0.2	1.25	3.7
	0.3	1.80	4.6
SS	0.1	8	11
	0.2	19	28
	0.3	25	48

[a] Assume T = 5 days

in. x 2.54 = cm
lb/acre x 1.121 = kg/ha



acre-in. x 1.029=ha-cm
lb/acre x 1.121 = kg/ha

Figure 19. Runoff volume versus pollutants in Des Moines [11].

TABLE 48. TIME WEIGHTED NORMALIZATION OF BOD AND SUSPENDED SOLIDS

	Suspended solids				BOD			
	0-0.5 h	0.5-1 h	1-2 h	>2 h	0-0.5 h	0.5-1 h	1-2 h	>2 h
Combined sewer overflows								
District of Columbia [32, 33, 34]	1.0	1.07	0.91	0.88	1.0	0.73	0.47	0.30
Milwaukee, Wisconsin [21]	1.0	0.78	0.55	0.55	1.0	0.90	0.61	0.38
Racine, Wisconsin [19]	1.0	0.78	0.47	0.34	1.0	0.67	0.60	0.38
Rochester, New York [20]	1.0	0.78	0.67	0.44	1.0	0.66	0.43	0.34
San Francisco, California [22]	1.0	0.77	0.80	0.44	1.0	0.56	0.41	0.27
Storm sewer outfalls								
District of Columbia [32, 33, 34]	1.0	0.59	0.48	0.12	1.0	0.93	1.23	0.46
Racine, Wisconsin [19]	1.0	0.60	0.26	1.43[a]	1.0	0.69	0.66	1.82[a]
Durham, North Carolina [12]	1.0	0.92	0.73	0.57	1.0	0.36	0.39	0.20
San Francisco, California [22]	1.0	0.31	0.37	0.15	1.0	0.30	0.14	0.09
Tulsa, Oklahoma [14]	1.0	0.55	0.33	0.93	1.0	0.72	0.64	0.60

Note: Values indicate relative pollutant concentrations as fractions of the concentration for the initial time interval.

a. Values based on only one sample.

RESIDUALS

The treatment and disposal of sludges is often the most difficult and costly portion of a water pollution control system. The solids generated by stormwater treatment systems must be carefully considered when designing control facilities. The assessment includes three principal areas:

1. Characterization of the solids by quantity, concentrations, and mass loading
2. Sludge thickening and dewatering
3. Final disposal of sludges

Characterization

A limited amount of work has been done to investigate the properties of stormwater and combined sewerage treatment sludges. The pollutant concentrations of samples reported by one investigation are shown in Table 49 [36].

TABLE 49. CHARACTERISTICS OF SLUDGE FROM COMBINED SEWER OVERFLOW TREATMENT [35]
mg/L

Location	SS	VSS	BOD	Total phosphorus as P	Total Kjeldahl nitrogen as N
Boston, Massachusetts	110 000	41 400	12 000	293	28
Kenosha, Wisconsin	8 300	5 225	1 700	194	492
Milwaukee, Wisconsin					
Hawley Road	41 900	10 570	3 200	149	517
Humboldt Avenue	17 400	8 425	2 200	109	56
New Providence, New Jersey					
Primary, wet weather	1 215	780	728	22	65
Primary, dry weather	3 840	3 200	1 600	41	214
Secondary, wet weather	25 070	14 770	11 200	436	6
Secondary, dry weather	4 620	3 610	2 950	93	277
Philadelphia, Pennsylvania	7 000	1 755	......	12	46
Racine, Wisconsin	8 433	3 340	1 100	39	112
San Francisco, California	22 500	8 850	1 000	166	375

The samples represent several different types of stormwater treatment sludges and, although there are not enough samples to indicate definite relationships, some observations about solids concentrations can be made. Two samples were taken from sedimentation basins in Milwaukee (Humboldt Avenue) and Boston (Cottage Farm). The suspended solids concentrations ranged from 1.74 to 11% with a volatile percentage of 38 to 48%. The large variation in concentrations is probably due to basin design and settling time as well as physical differences in combined sewer overflow characteristics. The samples from Philadelphia and Racine are screen backwashings and backwashings with some flotation scum, respectively. They show lower solids concentrations than the sedimentation sludges and the volatile portion is only 25 to 40%. The San Francisco and Milwaukee Hawley Road samples have been taken from the float portion of dissolved air flotation tanks. They show a solids concentration from 2.25 to 4.19%. The final two sets of samples came from biological processes at Kenosha and New Providence. The wet-weather samples had a solids concentration range of 0.12 to 2.5% but a high volatile fraction.

Although different designs for the stormwater control facilities would affect the solids concentrations, it appears that sedimentation produces the most concentrated sludge with flotation, biological process, and screening sludges becoming more dilute in that order.

Sludge Thickening and Dewatering

The treatability of stormwater solids was also studied by Envirex [3]. Cost-effective sludge handling and disposal requires that the solids be easily thickened for digestion or dewatered for incineration or land disposal. The results of the laboratory scale concentration tests are given in Tables 50 and 51 and summarized in Table 52. The results show that most of the sludges can be concentrated by conventional techniques.

TABLE 50. THICKENING OF SLUDGE FROM COMBINED SEWER OVERFLOW TREATMENT [35]

	Gravity thickening					Flotation thickening				
	Raw sludge sample, % solids	Thickened sludge, % solids	Mass loading, $lb/ft^2 \cdot d$	Chemical used	Chemical dose, lb/1000 lb	Thickened sludge, % solids	Mass loading, $lb/ft^2 \cdot d$	Recycle, %	Chemical used	Chemical dose, lb/1000 lb
Boston, Massachusetts - sludge from combined sewer overflow storage basin	11.0	14	32	......	...	....	...	...	......	....
	4.4	....	....	......	...	7.2	40	570	105C[a]	0.56
Kenosha, Wisconsin - contact stabilization reaction tank	0.83	1.0	5	C-31[b]	12	3.1	20	190	C-31[b]	5.35
Milwaukee, Wisconsin										
Hawley Road - flotation sludge	3.65	10	34	......	...	13	60	380	3A3[c]	1.05
Humboldt Avenue - chemically clarified Sludge from storage basin	1.74	6	9	......	...	14	22	750	......	....
New Providence, New Jersey -										
Primary clarifier, wet weather	0.12	8.0	80	837-A[d] CaO	5 333	5.9	80	160	......	....
Primary clarifier, dry weather	0.38	5.0	4	......	...	4.0	10	230	......	....
Final clarifier, wet weather	2.5	4.0	4	$FeCl_3$ 905-N[e]	105 2	4.1	30	290	837-A[d]	0.29
Final clarifier, dry weather	0.46	2.0	5	......	...	8.0	20	290	......	....
Racine, Wisconsin - screen bachwash and flotation sludge	0.84	10	<400	......	...	....	...	...	......	....
	2.72	....	....	......	...	8	10	380	......	....
	6.9	....	....	......	...	19	80	185	......	....
San Francisco, California - flotation sludge	2.25	4.5	2	......	...	6.1	20	365	105C[a]	0.26

a.	Altasep 105C	cationic polyelectrolyte.
b.	Dow C31	cationic polyelectrolyte.
c.	Altasep 3A3	anionic polyelectrolyte.
d.	Magnafloc 837-A	anionic polyelectrolyte.
e.	Magnafloc 905-N	nonionic polyelectrolyte.

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TABLE 51. DEWATERING OF SLUDGE FROM COMBINED SEWER OVERFLOW TREATMENT [35]

Source of sample	Raw sample, % solids	Centrifuge				Vacuum filtration				
		Dewatered sludge, % solids	Solids recovery, %	Chemical used	Chemical dose, lb/1000 lb	Cake solids, %	Yield, lb/ft^2·h	Filtrate solids, %	Chemical used	Chemical dose, lb/1000 lb
Boston, Massachusetts - sludge from combined sewer overflow storage basin	11.0	34	92	105C[a]	0.18	..	..	.....	.....	......
Kenosha, Wisconsin - contact stabilization reaction tank	0.84	9	0	....	....	..	..	.....	.....	......
	2.59	7	0	....	....	..	..	.....	FeCl3	60
	3.10	....	..	....	....	15	4	82	CaO	128
Milwaukee, Wisconsin										
Hawley Road - flotation sludge	3.7	23	92	....	....	..	..	.....	3A3[b]	0.76
	9.9	30	95	3A3[b]	0.20	36	7	232	CaO	95
Humboldt Avenue - chemically clarified sludge from storage basin	1.74	32	95	C-41[c]	3.4	..	..	.....	.....	......
New Providence, New Jersey -										
Primary clarifier, wet weather	0.12	1.5	77	....	....	..	..	.....	FeCl3	54
	2.5	....	..	....	....	28	4	82	CaO	160
Primary clarifier, dry weather	1.75	13	92	....	....	..	..	.....	FeCl3	206
	2.6	....	..	....	....	22	7	68	CaO	38
Final clarifier, wet weather	2.5	8	92	FeCl3	1 458	..	..	.....	FeCl3	85
	3.15	....	..	905N[d]	....	19	4	231	CaO	254
Final clarifier, dry weather	0.46	2.6	90	FeCl3	216	..	..	.....	FeCl3	567
	2.6	....	..	....	....	14	2	45	CaO	212
Racine, Wisconsin - screen backwash and flotation sludge	7.5	33	96	....	....	..	..	.....	1A1[e]	0.74
	2.7	....	..	....	....	23	3	1 400	CaO	147
San Francisco, California - flotation sludge	2.25	11	93	105C[a]	0.53	18	2	123	CaO	444

a. Altasep 105C cationic polyelectrolyte
b. Altasep 3A3 anionic polyelectrolyte
c. Dow C41 cationic polyelectrolyte
d. Magnafloc 905-N nonionic polyelectrolyte
e. Altasep 1A1 anionic polyelectrolyte

lb/ft^2·h x 4.88 = kg/m^2·d
lb/1000 lb = kg/1000 kg

TABLE 52. SUMMARY OF CONCENTRATION OF COMBINED SEWER OVERFLOW TREATMENT SYSTEMS

Location	Treatment	Sludge volume/ volume of raw combined sewer overflow, %	Solids content of sludge, %	Sludge concentration (pilot studies) - solids content of sludge, %			
				Gravity	Flotation	Centrifuge	Vacuum filter
Boston, Massachusetts	Storage	0.2	4.4-11.0	14	6-8	30-35	....
Kenosha, Wisconsin	Activated sludge	3.5	0.83	1	3	9	15
Milwaukee, Wisconsin							
Hawley Road	Flotation	0.4	3.65	10	13	23-30	36
Humboldt Avenue	Storage	0.9	1.74	6	11-14	30-35	....
New Providence, New Jersey	Trickling filters	0.7					
Primary, wet weather			0.12	8	6	1.5	28
Primary, dry weather			0.38	5	4	13	22
Secondary, wet weather			2.50	4	4	7.5	18.5
Secondary, dry weather			0.46	2	8	2.6	14
Philadelphia, Pennsylvania	Microscreens	6.0	0.2	..	.....	.....	....
Racine, Wisconsin	Screens/ flotation	4.8	0.84	10	13	20-33	23
San Francisco, California	Flotation	0.4-0.7	2.25	4	6	11	18

Final Disposal of Sludge

There are two basic options for the disposal of sludge from stormwater treatment facilities:

1. Bleedback of solids to the dry-weather treatment plant using either the extra capacity of existing sludge disposal units or adding capacity to handle storm related loading.
2. Handling of the sludge at the site of stormwater treatment facility with a treatment system dedicated solely to stormwater solids.

Bleedback to the Dry-Weather Treatment Plant--

It is a common practice to transport stormwater solids to the sewage treatment plant after the storm event has subsided. This can be done either for the total flow from a holding basin or sludges from treatment facilities. The simplest transport method is to feed the solids into the sanitary sewer system or interceptor at a controlled rate. The two principal design considerations of a bleedback system are the capacity of the delivery system and the ability of the treatment plant to handle the additional mass loading. The effect on treatment plant capacity is illustrated in Example Problem 5-1.

The mechanics of sludge transport have been studied for conventional sewage treatment plants and can be applied to stormwater sludge. The solids concentration and flow velocity are the two most important parameters to consider. Sludge concentrations greater than 6% become difficult and expensive to pump. At low velocities, the solids will tend to settle out in the sewers (potentially restricting or clogging the lines) and may be resuspended during high flow periods causing slug loading at the treatment plant. The basic flow mechanics principles can be used to investigate existing sewer lines to determine if bleedback into the sanitary sewer system is feasible or if a dedicated sludge main is required.

In addition to potential overloading of a dry-weather treatment plant with storm flows, it is possible that toxic elements in runoff could disrupt biological processes such as secondary treatment and sludge digestion. Storm runoff may contain greater concentrations of certain toxic substances than commonly found in domestic sewage, and the addition of runoff or stormwater treatment sludge to a wastewater treatment plant may be toxic to the biological treatment organisms. It is difficult to pinpoint the levels of metal concentrations that will decrease biological activity. Metals can have complex synergistic effects; biological organisms vary in their susceptibility; and biomasses may become acclimated to low metal concentrations. Several investigators have studied metal toxicity in biological sewage treatment systems and the results are shown in Table 53. The concentrations shown are the levels at which reduced efficiency was noted in the treatment system indicated.

The expected concentrations of metals in sludges from combined sewage treatment facilities was tabulated in reference [36] and is reproduced as Table 54. Comparison of Tables 53 and 54 indicates that the raw sludge may be

EXAMPLE PROBLEM 5-1: IMPACT OF STORMWATER TREATMENT SOLIDS ON THE DRY-WEATHER TREATMENT PLANT

Determine the bleedback loading to a dry-weather plant from a satellite stormwater treatment facility for a 1 in. rainstorm and compare the loading to average plant loads.

Specified Conditions

1. Tributary area population = 50 000
2. Average dry-weather flow = 150 gal/capita·d
3. Average suspended solids concentration = 200 mg/L
4. Combined sewer area = 5 000 acres

Assumptions

1. Fifty percent of the rainfall over the area will run off and overflow to the stormwater treatment facility.
2. The average suspended solids concentration of the overflow = 300 mg/L.
3. The stormwater treatment facility will capture 50% of the overflow solids.
4. The captured solids will be pumped to the dry-weather treatment facility headworks at a solids concentration of 2%.
5. The dry-weather plant will operate satisfactorily at an overloading of 125% of average flow.

Solution

1. Compute average dry-weather flow and solids loading

Flow = 50 000 people x 150 gal/capita·d
= 7.5 Mgal/d
SS = 7.5 Mgal/D x 200 mg/L x 8.34 lb/gal
= 12 500 lb/d

2. Compute volume of stormwater treated at the satellite stormwater treatment facility.

Volume = 1.0 in. rain x 0.5 x 5 000 acres x 0.027 Mgal/acre·in.
= 66.7 Mgal

3. Compute mass and volume of stormwater treatment sludge pumped to the dry-weather plant.

Mass = 66.7 Mgal x 300 mg/L x 8.34 gal x 0.50 capture
= 83 000 lb
Volume = 83 000 lb 0.02 solids concentration 8.34
= 0.50 Mgal

4. Comparison

	Flow	Solids
Average-dry weather conditions	7.5 Mgal/d	12 500 lb/d
Overflow from a 1 in. storm	0.50 Mgal	83 000 lb

Comment

The problem shows that the runoff from a basin can be reduced to a manageable flow quantity with satellite stormwater treatment but the solids will present a serious problem for bleedback. The dry-weather plant could handle the flow in its 25% buffer capacity over a period of 7 hours; however, the solids would need more than 25 days of bleedback to be treated in the spare capacity. Obviously, the 1 in. storm is far too large to be treated and bleedback to an existing dry-weather facility. Expanded treatment units or large storage basins would have to be available.

potentially toxic to aerobic treatment systems but not to anaerobic digestion. The potential toxicity must be examined for each installation to determine the actual concentration of metals in the treatment plant effluent. In most cases, the sludge would be combined with sanitary sewage that had low metal concentrations. The resulting diluted influent would not be toxic to the treatment organisms. However, bleedback operations will usually take place during periods of low sanitary sewage flow, when the available hydraulic capacity of the treatment plant is the largest, but dilution capacity is low.

TABLE 53. CONCENTRATIONS OF METALS REPORTED TO CAUSE REDUCED EFFICIENCY IN BIOLOGICAL TREATMENT SYSTEMS
mg/L

Reference No.	Silver	Nickel	Copper	Chromium	Zinc
Aerobic systems, raw sewage					
[37]	10	10	25	25	100
[38]	..	1-2.5	1	10	5-10
[39]	..	2-2.5	1	1	2-5
Anaerobic digestion, sludge					
[38] (primary)	..	62	280	330	375
[38] (waste activated)	..	89	160	530	328
[40]	..	500	14-150	500	100-1 000

TABLE 54. METAL CONCENTRATIONS IN VARIOUS COMBINED SEWAGE OVERFLOW TREATMENT SLUDGES [36]
mg/L

Treatment process	Nickel	Copper	Chromium	Zinc	Lead	Mercury
Storage alone	0.1	1.5	0.05	0.6	0.7	0.001
Storage/sedimentation	2.5	8.4	4.4	15.2	29.0	0.05
Dissolved air flotation	2.3	10.0	45.6	19.4	43.3	0.11
Screening/dissolved air flotation	1.8	4.1	1.8	13.8	8.6	0.02
Microscreening	2.0	1.4	0.4	8.3	17.1	0.01
Contact stabilization	5.3	14.5	17.3	71.5	5.3	0.03
Trickling filter	25.2	32.8	79.5	41.7	11.4	.....

Bleedback of stormwater treatment sludges to a sewage treatment plant has the advantage of locating all of a district's sludge disposal operations at one site with maximum utilization of reserve capacities. However, stormwater sludges cannot be neglected and the loadings must be considered in the design of or additions to the treatment plant.

Handling of the Sludge at the Site of the Stormwater Treatment Facility--

If it is necessary to maintain sludge handling facilities at stormwater treatment sites, the characteristics of the sludges should be carefully considered in facility design. Basically, the sludge will have to be dewatered and disposed of by a treatment system similar to ones used for municipal sewage sludge. The intermittent nature of storm events, and consequently stormwater sludge, will probably rule out some common processes for remote facilities that treat only stormwater sludges. Biological systems, such as digestion, that require a continuously fed biomass and other systems, such as incineration, that would require extended startup times would not be used at stormwater treatment sites. Thickening, vacuum filtration, pressure filtration, and centrifugation would all be acceptable means of volume reduction and the best options for disposal are either land filling or land spreading, preceded by a heat or chemical stabilization step to decrease nuisance and health hazard potential. There is little existing design experience to indicate that any of these common dewatering or disposal techniques are particularly suited to stormwater solids treatment.

RECEIVING WATER IMPACTS

The goal of a stormwater runoff study is to evaluate the impact of runoff and combined sewer overflows on the receiving waters and decide what control alternatives would be most cost effective in reducing wet weather pollution. An evaluation of the way in which receiving water characteristics are influenced by stormwater runoff is always difficult to perform because of the masking effect of municipal and industrial point sources, because runoff events are both intermittent and highly variable, and because of carryover effects of stormwater benthic deposits. The impact of a storm varies with rainfall volume, duration, intensity, and the antecedent conditions of the basin.

The methodology frequently used to study the impact of stormwater on streams, lakes, and estuaries is to model the characteristics of runoff for a variety of storm conditions and input the resulting mass loadings into a receiving water model. A modeling approach allows the investigator to study a large variety of storm and stream conditions that probably would not occur during the time frame of the project. Modeling also allows the study of one of the many variable influences while keeping the remaining ones constant. The methodology for modeling stormwater pollutants has been presented in earlier sections of this report. Other methods include direct measurement and simple correlations.

Obviously, impacts are often very site specific and the extent of the problems will depend heavily on local conditions, such as rainfall quantities, point sources of pollution and their treatment, land use, and the sensitivity of the receiving water. Urban stormwater pollution can be manifested in a number of ways depending on the specific factors of the locality being studied. Individual site conditions will influence both the mass loading of pollutants and the ability of the receiving water to assimilate the loading. Problems result when loadings exceed the assimilative capacity of a stream or lake and the use of the water is impaired. The classes of problems that may be caused can be broadly categorized as follows [41]:

- Aesthetic deterioration and solids - Either general appearance (dirty, turbid, cloudy) or the actual presence of specific, objectionable conditions (odors, floating debris, oil films, scum or slimes, etc.) may make the receiving water unattractive or repugnant to those in its proximity. In addition, particulate matter may cause the formation of sediment deposits that smother bottom dwelling aquatic organisms or restrict river flows contributing to flooding potential. Excessive solids can also make the receiving water an unacceptable source for agricultural irrigation water.

- Dissolved oxygen depletion - Organic materials stimulate the growth of bacteria which may consume oxygen faster than natural processes can replenish. This condition may or may not be visually apparent. In the extreme, discoloration, gas formation, and odors may be apparent--however, well before this extreme is reached, conditions suitable for a balanced aquatic population of fish and lower species in the food chain may be violated. The presence of unoxidized nitrogen compounds (e.g., ammonia) is in some cases a significant element in water quality problems related to low dissolved oxygen levels.

- Pathogen Concentrations - The presence of excessive concentrations of objectionable microorganisms can impair the ability to utilize the receiving water for certain water supply and recreational purposes.

- Nutrients - The discharge of materials which fertilize or stimulate excessive or undesirable forms of aquatic growth can create significant problems in some receiving water systems. Overstimulation of aquatic weeds or algae (eutrophication) can be aesthetically objectionable, cause dissolved oxygen problems, and in extreme cases, can interfere with recreational use and create odors and heavy mats of floating material at shorelines.

- Toxicity - Toxicity problems can fall into either of two categories: (1) metals/pesticides/persistent organics, which may exhibit a subtle, long-term effect on the environment in areas well removed from the area under consideration by the discharge of small quantities which gradually accumulate in sensitive areas, and (2) ammonia and byproducts of effluent chlorination which, under some conditions, can exhibit a local, more immediate impact.

Dissolved Oxygen Depletion

The classical problem related to organic pollution of receiving waters is the consumption of instream oxygen by the bacterial breakdown of organic material. The resulting low levels of oxygen will destroy sensitive species of fish and aquatic organisms. The organic material (and unoxidized nitrogen compounds) in runoff can be important to the oxygen balance of streams.

Colston studied the dissolved oxygen (DO) sag for a watershed in North Carolina by analyzing several storm types and intervals during storms. The results of the study are presented in Table 55.

TABLE 55. RESULTS OF OXYGEN SAG COMPUTATIONS FOR DURHAM, NORTH CAROLINA [12]

Storm type	Rainfall, in.	Storm component	Storm flow, ft^3/s	BOD$_{20}$, mg/L	DO at sag point, mg/L
Small	0.1	Total	40	40	10.0
Small	0.1	Total	20	31	10.0
1 to 2 yr storm	1.0	First flush	200	75	4.5
		Peak	315	62	3.8
		Falling limb	200	47	6.5
		Tail	75	37	8.7
5 yr storm	3.3	First flush	500	85	0
		Peak	1 100	70	0
		Falling limb	800	54	0.3
		Tail	300	42	5.9
7 d, 10 yr low flow	...	....	0.3	15	0

in. x 2.54 = cm
ft^3/s x 28.316 = L/s

The analysis shows that the severity of DO depletion increases with the size of the storm and is most severe for the slug of runoff at the peak of the storm. The study was performed for the Third Fork Creek and the river characteristics and reaeration coefficients are necessarily site specific.

In a study for the Corps of Engineers the impact of urban runoff from basins in the Atlanta, Georgia, area was analyzed [10]. The effects of a 1.02 cm (0.4 in.) storm on a highly urbanized basin were plotted to determine the impact on dissolved oxygen levels in the Chattahoochee River. The DO profile in Figure 20 shows that the DO decreased to a level of 1.5 mg/L approximately 48.3 km (30 mi) downstream from the addition of runoff. The level is below the state standard of 5 mg/L of dissolved oxygen. Estimates from the study indicate that this urban basin alone will cause a violation of stream standards during 50% of the rainfall events.

Pathogen Concentrations

Excess concentrations of bacterial indicator organisms in urban runoff will prevent water supply and recreational use of the receiving water. Although stormwater runoff should not contain fecal contamination, several investigators have measured significant levels of contamination. The sources are probably faulty septic tanks, illegal cross-connections, and contamination by domestic animals. The results of some investigations are shown in Table 56 [2, 42, 43].

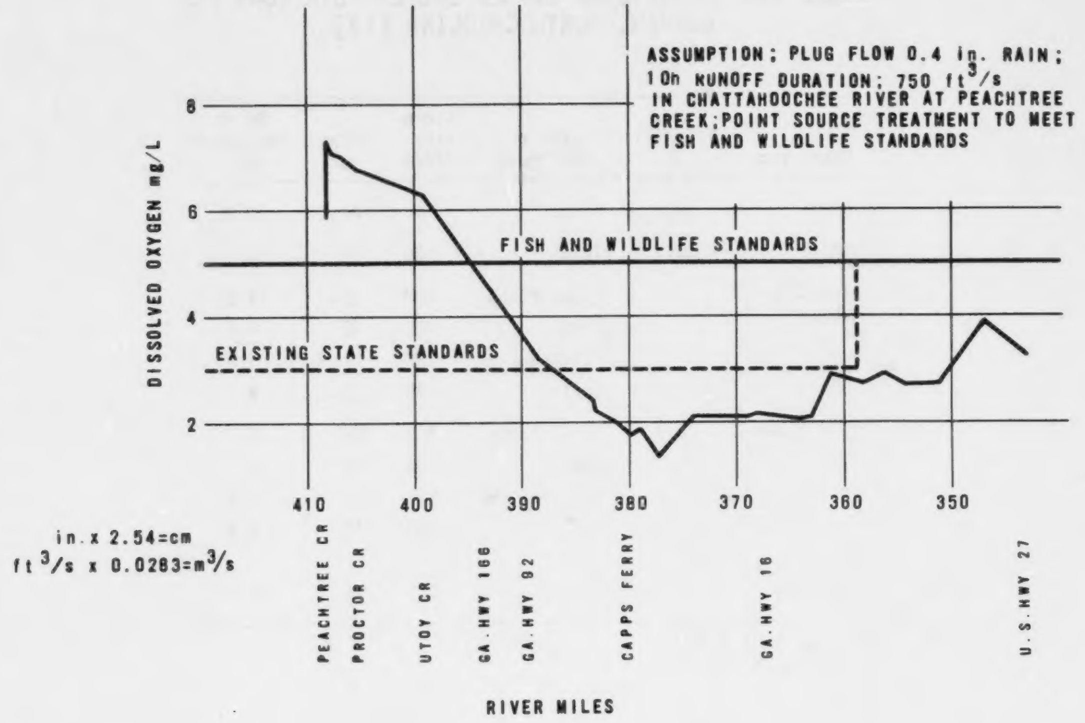


Figure 20. Impact of urban runoff on the Chattahoochee River [10].

TABLE 56. REPORTED BACTERIAL CONTAMINATION OF STORMWATER POLLUTANTS
Organisms/100 mL

	Total coliforms	Fecal coliforms	Fecal streptococci
Business district [42]	......	13 000	51 000
Residential [42]	......	6 500	150 000
Rural [42]	......	2 700	58 000
Cincinnati, residential [2]	58 000	10 900	20 500
Miami [43]			
Residential	5 100	3 600	700
Parking lot	50 500	49 800	1 200
Residential	19 400	16 400	1 100

A common bacterial standard for recreational use of water is a total coliform count of less than 1 000 organisms per 100 millilitres and a fecal coliform level of less than 200 organisms per 100 millilitres. Looking at Table 56, it is clear that stormwater runoff will contaminate the receiving water at an outfall and, depending on diffusion and dilution, may make adjacent areas unacceptable for water contact recreation.

Nutrients

The influx of nutrient materials into a body of water will fertilize and stimulate the aquatic weeds and algae. The function of urban runoff in supplying the excess nutrients to receiving waters has to be examined in an effort to curtail lake eutrophication.

The Lake Wingra watershed in Madison, Wisconsin, was investigated to compute the source of nutrient loading. A tabulation of nutrient sources is given in Table 57.

TABLE 57. NUTRIENT SOURCES FOR LAKE WINGRA [44]
lb/yr

Source	NH_3-N	NO_3-N	Org-N	Soluble P	Total P
Precipitation	860	880	570	55	70
Dry fallout	1 230	1 060	2 420	46	240
Flow from springs	370	9 120	.....	66	170
Urban runoff	990	1 320	7 710	1 120	2 160

lb/yr x 0.0703 = kg/yr

The urban runoff is an important part of the mass balance especially for the phosphorus loading. More than 80% of the influent phosphorus comes from runoff.

An analysis of potential phosphorus loadings on Atlanta area reservoirs was made to evaluate the eutrophic potential. The Atlanta data are summarized in Table 58. The estimated loadings of phosphorus from only urban runoff were compared to permissible loading rates developed by Vollenweidor in a worldwide study of eutrophic lakes [10].

Several of the reservoirs will be exceeding the permissible phosphorus limits and, depending upon nitrogen and light availability, will be expected to experience some degree of eutrophication.

Toxicity

Urban runoff can be an important source of the metals and pesticides that can be toxic to aquatic life. It is very difficult to isolate the instream impacts of toxic elements from urban runoff and very little work has been attempted. One study did quantify the amount of potential toxins on street surfaces to help analyze the potential for adverse impact. A summary of this ten city study that analyzed the components of street solids is shown in Table 59.

TABLE 58. URBAN RUNOFF PHOSPHORUS LOADINGS COMPARED TO THE POTENTIAL FOR LAKE EUTROPHICATION [10]
lb/1000 ft^3

	Projected phosphorus loadings		Vollenweider's loadings	
Reservoir	1980	2000	Permissible	Dangerous
Morgan Falls	0.246	0.448	0.18	0.34
Jackson Lake	0.185	0.257	0.05	0.10
Stone Mountain	0.046	0.063	0.04	0.07
Lake Allatoona	0.011	0.180	0.03	0.06
Lake Spivey	0.002	0.004	0.05	0.09
Clayton County Water intake	0.025	0.045	0.02	0.04

lb/1 000 ft^3 x 0.0160 = kg/m^3

TABLE 59. POTENTIALLY TOXIC ELEMENTS IN STREET SURFACE SOLIDS [1]

Element	Loading, lb/curb mi
Chromium	0.11
Copper	0.20
Zinc	0.65
Nickel	0.05
Mercury	0.73
Lead	0.57
Cadmium	0.003
Total heavy metals	1.6
Total pesticides	$1\,420 \times 10^{-6}$
PCB	$1\,100 \times 10^{-6}$

lb/curb mi x 0.2819 = kg/curb km

Although the street surface loadings are not a precise measurement of the quantities that can wash off in a storm event, it is an indication of the reservoir of toxins available.

In his study in North Carolina, Colston measured the concentrations of metals in stormwater runoff samples. The ranges of values detected for some of the metals are presented in Table 60.

TABLE 60. CONCENTRATIONS OF METALS IN URBAN RUNOFF
FROM DURHAM, NORTH CAROLINA [12]
mg/L

		Range	
Pollutant	Mean	High	Low
Chromium	0.23	0.47	0.06
Copper	0.15	0.50	0.04
Lead	0.46	2.86	0.1
Nickel	0.15	0.29	0.09
Zinc	0.36	4.6	0.09

Relative Quantities of Urban Stormwater Pollutants

The signficance and relative importance of urban runoff as a source of pollution can be demonstrated from the work of Colston in Durham, North Carolina [12]. His study resulted in the comparison shown in Table 61.

TABLE 61. COMPARISON OF RAW MUNICIPAL WASTE
AND URBAN RUNOFF [12]
lb/acre·yr

Pollutant	Raw municipal wastes	Urban runoff
COD	1 027	938
BOD_5	685	470
Suspended solids	335	6 690
Kjeldahl nitrogen as N	....	6.1
Nitrate as N	7.2	....
Total phosphorus as P	11	4.7
Chromium	0.10	1.6
Copper	0.20	1.6
Lead	0.08	2.9
Nickel	0.16	1.2
Zinc	1.5	2.0

lb/acre·yr x 1.21 = kg/ha·yr

The results indicate that even with an acceptable level of treatment for the municipal waste, the water quality of the receiving stream would be degraded by urban runoff. During periods of wet weather, the stream quality is controlled by the urban runoff characteristics, not municipal sewage loadings.

Instream characterization of the chemical quality of the Roanoke River during both wet- and dry-weather periods also indicates the importance of runoff on water quality. The results are tabulated in Table 62.

TABLE 62. POLLUTANT CONCENTRATIONS IN ROANOKE RIVER TRIBUTARIES DURING WET- AND DRY-WEATHER CONDITIONS [45]
mg/L

Pollutant	Murray Run	Trout Run	24th Street
BOD			
Dry weather	8	3	8
Wet weather	17	18	20
Total solids			
Dry weather	248	281	194
Wet weather	623	460	514
Total volatile solids			
Dry weather	85	147	126
Wet weather	134	139	172
Suspended solids			
Dry weather	37	17	20
Wet weather	89	93	103
Volatile suspended solids			
Dry weather	12	8	7
Wet weather	25	28	24

As shown, the solids and organic levels increase at least two-fold for periods of runoff.

A third study in Metropolitan Seattle also surveyed two urban creeks to compare quality constituents for dry- and wet-weather periods [46]. Harper found the following increases in concentration during the wet season: BOD, 4 mg/L; PO_4, 0.5 to 1 mg/L; total solids, 20 to 30 mg/L; and turbidity, 10 to 30 FTU. Zinc levels increased significantly in both streams while lead and copper increases were significant in one of the streams. Although the BOD levels were higher during the wet season, the dissolved oxygen concentrations in the streams did not decrease. The authors credit this fact to an increased rate of stream reaeration during storm periods due to higher stream velocities and increased turbulence. The BOD-dissolved oxygen relation shows the importance of the nature of the receiving water in evaluating pollutant impacts. A more placid stream or river may have exhibited a DO sag due to the BOD loading and less reaeration. The ultimate receiving water should also be considered in a regional analysis; in this case, the urban runoff may have caused some DO depletion near the stream mouths in Puget Sound.

In addition to the chemical constitutents of the Seattle creeks, the benthic communities were studied and compared to a clean water creek. The results indicate a lower quantity of sensitive organisms and a poorer diversity index in the urban streams. The inhibited nature of the benthic community is attributed to a combination of sedimentation, scouring, and chemical toxicity from metals, oil, and grease.

The high variations in flow in urban streams may be the most important deterrent to the development of a stable community. The increasing imperviousness of the developing area causes an increase in runoff volume and a faster stream reaction to precipitation. The base flow of a stream decreases and storm peaks increase. The stream fauna cannot adapt to highly variable flows, scour, and increased sediment.

SECTION 6

BEST MANAGEMENT PRACTICES FOR STORMWATER POLLUTION CONTROL

Much emphasis is currently being placed on controlling stormwater pollution by attacking the problem at its source, as opposed to potentially more costly downstream treatment facilities. These source controls, termed "Best Management Practices" (BMP), are a practice or combination of practices that are determined to be capable of being implemented and most effective in reducing the amount of pollution generated by a nonpoint source to a level compatible with water quality goals.

Best Management Practices are classified into two groups: (1) planning, where efforts are directed to the control of future development or redevelopment of existing areas, and (2) maintenance and operational practices to reduce the impact of nonpoint source contamination from existing developed areas.

Successful stormwater pollution control depends on the effective implementation of the proposed planning efforts and/or control practices. Legislation or ordinances, to force or encourage conformance with the intended BMP, has been found most effective in achieving this end. Essential to successful implementaion and enforcement is a concerted effort to monitor compliance with the intended legislation and educate not only those who will bear the responsibility of regulation, but the public as well.

PLANNING

The concept of preventing and reducing the source of stormwater pollution best applies to developing urban areas, for these are areas where man's encroachment is yet minimal, or at least controllable, and drainage essentially conforms to natural patterns and levels. Such lands, in consequence, offer the greatest flexibility of approach in preventing pollution. What is required, therefore, is to manage development in such a way that a runoff regime may be retained close to natural levels. It is in these new areas where proper management can prevent long-term problems.

The goal of planning is to develop a macroscopic management concept to prevent the problems resulting from short-sighted development of individual areas. When considering stormwater management, the planner is interested in controlling the volume and rate of runoff as well as the pollutional characteristics. The goal is to preserve the initial ecological balance so that expensive downstream treatment facilities can be minimized. Since the size of storm sewer networks and treatment plants is quite sensitive to the flow, quantity, and particularly the peak flowrate, a reduction in total volume or a smoothing out of the peaks will result in lower construction costs.

Land Use Planning

The starting point of land use planning is the knowledge that traditional urbanization upsets the natural hydrologic and ecologic balance of a watershed. The degree of upset, and whether it is beneficial or detrimental, depends on the mix, location, and distribution of the proposed land use activities. As man urbanizes an area, the receiving waters are degraded by runoff from his activities. Effective planning requires that limits be put on the stream degradation and that the quantitative effects of development options be evaluated to weigh their merits and decide what restrictions should be placed on the residuals emitted from each site.

Computer simulation is an important planning tool for examining the interacting pollutant sources in a watershed. By modeling the runoff process for urbanizing areas, a planner can predict the effects of proposed plans and the ability of controls to solve potential problems. The receiving water system and point sources of pollution should be included in the evaluation to understand the relative importance of urban runoff. Several existing models that can be used to examine the runoff process have been described in Section 4. Water quality criteria standards can be recommended after investigating the sources of pollution and the ability of the receiving water to absorb loadings.

Having set goals for the watershed, the planning agency has two basic choices for achieving the water quality standards. Either the individual sites can be forced to comply with individual practices and performance standards that fit into the master plan, or the basin system can be designed and maintained as a public utility. The decision on how to blend the options to meet specific site conditions is the key to implementing a basin plan. Isolated development tracts can be controlled by requiring developers to follow specific source control practices, or a simple set of performance standards can be applied and the choice of practices can be left up to the developer. For example, the agency can require that the runoff from the developed site must not exceed predevelopment intensity. The developer will have to minimize runoff producing areas and provide detention facilities at the site.

When dealing with individual or small-scale construction in an urbanizing area, a public utility must ensure that stormwater control planning is implemented. The utility needs the power to acquire land to preserve natural floodways and infiltration areas before development overruns the best sites. Dealing with the small-scale development is a difficult political problem when stressing nonstructural controls. Plans must be developed, and specific sites must be set aside for greenways, detention ponds, and floodways before urbanization begins. This involves buying the land or inverse condemnation before the tax base has been developed to pay for it.

Planners also must consider the effects of their actions on areas outside the individual watershed. For example, detaining storm flow in a downstream watershed while it remains unregulated upstream can cause higher flood levels in the river than a completely unregulated system.

Use of Natural Drainage Features

The traditional urbanization process upsets the existing water balance of a site by replacing natural infiltration areas with roadways, parking lots, roofs and other impervious areas. The increased quantity of runoff is carried away in concrete culverts or compacted earth channels instead of in natural channels and grassy floodways. The net impact is increased runoff, decreased infiltration to the groundwater, and increased flowrates. The increased flow velocities will mean increased channel erosion and the transport of surface material to receiving waters. Although most of the surface material is natural and harmless on the land, it will become a water pollutant contributing to stream degradation. If the natural drainage features can be preserved, flowrate increases will be minimized and pollution loads contained.

The key to preserving a natural drainage system for an urbanizing area is understanding the predevelopment water balance and designing to minimize interference with the system. The soils and hydrology of the site must be studied so that high-density, highly impervious construction, such as shopping centers and industrial complexes, is located in areas with naturally low infiltration potential, and the best recharge areas are preserved as open, undisturbed space in parks and woodlands. Runoff from developed areas should be directed to the recharge areas and detained to make the best use of the full infiltration potential. Any necessary drainage channels should be modeled on the natural swales of the undeveloped site. The broad, grassy swales will slow down the runoff and maximize infiltration. The drainage plan can include variable depth detention ponds that will rise during a runoff event and return to a base level during dry weather.

Realizing that the goal of the design is maximizing infiltration-recharge and minimizing runoff, the planner should be able to incorporate the following techniques into the site plan:

- Roof leaders should discharge to pervious areas or seepage pits.
- As much area as possible should be left in a natural undisturbed state. Earthwork and construction traffic compact the soil and decrease infiltration.
- Steep slopes should be avoided. They will contribute to erosion and lessen recharge.
- Large expanses of impervious area should be avoided. Parking lots can be built in smaller units and drained to pervious areas.
- No development should be permitted in flood plains.

An interesting technological answer to the problem of preserving pervious area is the possibility of using an open graded asphaltic concrete as a paving material. Experiments have shown that it will serve as a porous pavement, allowing as much as 64 cm/h (25 in./h) of stormwater to infiltrate through the pavement [1].

Preliminary investigations have shown promise that it can withstand stability, durability, and freeze-thaw tests, and that it is comparable in cost to conventional paving with drainage. Long-term tests will have to be made of its resistance to clogging and the effects on the quality of water that filters through the pavement. If the soil under the pavement and base is free draining, the rainwater will infiltrate quickly into the ground; however, porous pavement can also serve as a ponding device if storm quantities exceed soil capacity. The porous nature of the pavement permits water to be stored in the pavement. A pavement with a 10 cm (4 in.) surface course and 15 cm (6 in.) base course could store 6.1 cm (2.4 in.) of runoff in its voids. The proven use of porous pavement can be an important tool in preserving natural drainage.

If natural drainage techniques are developed at a site, the resulting system should provide a water balance closely approximating the predevelopment conditions. The site will be less densely populated than most planned areas; however, the planner will have a community that should be more desirable to live in. In addition, it has been estimated that a natural drainage system costs about $1500 less per hectare ($600 less per acre) than a conventional system [2].

Erosion Controls

The control of erosion from construction and developing sites will have a major impact on the total pollution loads imposed on receiving waters. Current estimates indicate that approximately 3900 km^2 (1500 mi^2) of the United States is urbanized annually. All of this land is exposed to accelerated erosion.

From a knowledge of erosion and the guidelines that have been written concerning erosion control, several basic principles for control of erosion are apparent:

- Reduce the area and duration of soil exposure.
- Protect the soil with mulch and vegetative cover.
- Reduce the rate and volume of runoff by increasing infiltration rates and surface storage and by planned diversion of excess runoff.
- Diminish runoff velocity with planned engineering works.
- Protect and modify drainage ways to withstand concentrated runoff resulting from paved areas.

- Trap as much sediment as possible in temporary or permanent sedimentation basins.

- Maintain completed works and assure frequent inspection for maintenance needs.

These principles can be implemented by a variety of simply constructed facilities. Detailed descriptions and design criteria are available in the literature [3]. Costs for some of the basic erosion control alternatives are presented in Table 63. Problem assessment and effectiveness analyses have been performed, in order-of-magnitude terms, using the Universal Soil Loss equation described in Section 5.

TABLE 63. EROSION CONTROL COSTS PER DEVELOPED ACRE [4]

Vegetative measures	Initial placement cost, $/acre	First year maintenance cost, $/acre
Seeding: seedbed preparation, seed and application, mulching at 2 tons/acre		
Temporary seeding by machine	240-330	50-120
Temporary seeding by hand	335-415	50-120
Permanent seeding by machine	790-1 220	50-120
Sodding, including seedbed preparation	2 400-3 600	240-2 900
Mulch, 2 tons/acre		
By hand	120-140	---
By machine	90-120	---
Mechanical measures		
Earth diversion berms	0.15-0.30	1.20-3.60
Straw bale barriers	0.75-1.10	1.20-3.60
Silt basins with earth dam, watershed area		
2 acres to 5 acres	600-1 200	500-750
25 acres to 100 acres	1 200-3 500	750-1 200
100 acres to 200 acres	3 500-5 000	1 200-1 800

$/acre x 2.469 = $/ha
acre x 0.405 = ha
tons/acre x 2240 = kg/ha

MAINTENANCE AND OPERATION PRACTICES

The proper maintenance and cleanliness of the entire urban area can have a significant impact on the quantity of pollutants washed from an area by stormwater. Cleanliness of an urban area starts with control of litter, debris, deicing agents, and agricultural chemicals, such as pesticides and fertilizers. Regular street repair and sweeping can further minimize the pollutants picked up in stormwater runoff. The proper use and maintenance of both catchbasins and the collection system can maximize control of pollutants by directing them to treatment or disposal.

Neighborhood Areas

Litter control--

Spent containers from food and drink, cigarettes, newspapers, sidewalk sweepings, lawn trimmings, and a multitude of other materials carelessly discarded become street litter. Unless this material is prevented from reaching the street or is removed by street cleaning equipment, it often is found in stormwater discharges. Enforcement of antilitter laws, convenient location of sidewalk waste disposal containers, and public education programs are just some of the source control measures. While difficult to measure, the benefits that occur are aesthetic improvement of the urban area and reduced pollution of the urban runoff.

According to a recent California study [5], in urban areas, litter accumulated at a rate of approximately 1.8 kg/person·yr (4 lb/person·yr). Of this total, about 0.84 kg/person·yr (1.8 lb/person·yr) appears as litter between the curb lines of streets in urban areas. It was reported that about 21% of the material picked up during mechanical street sweeping was litter.

Chemical Use Control--

One the the most often overlooked measures for reducing the pollution from stormwater runoff is the reduction in the indiscriminate use and disposal of toxic substances such as fertilizers, pesticides, oil, gasoline, and detergents.

Operations such as tree spraying, weed control, and fertilization of parks and parkways by municipal agencies, and the use of pesticides and fertilizers by individual homeowners can be controlled by increasing public awareness of the potential hazards to receiving waters, and providing instruction as to proper use and application. In many cases over-application is the major problem, where use in moderation would achieve equal results. The use of less toxic formulations is another alternative to minimize potential pollution. Comparative toxicities for several organic phosphorus and chlorinated hydrocarbon insecticides have been presented [6].

Pesticides have been detected in samples taken from several urban areas with typical loadings, including PCBs, between 40 and 3400 mg/curb·km (0.000136 to 0.012 lb/curb·mi) [7]. Direct dumping of chemicals, crankcase oil, and debris into catchbasins, inlets, and sewers is a significant problem that may only be addressed through educational programs, ordinances, and enforcement.

Street Sweeping--

Street sweeping is used by most cities to remove accumulated dust, dirt, and litter from street surfaces, but cleaning is usually done for aesthetic reasons. In many neighborhoods the amount of paper tolerated by the public governs cleaning frequencies [6]. Street cleaning practices have been shown to be an effective method of attacking the source of stormwater-related pollution problems.

Street cleaning effectiveness is a function of (1) sweeper efficiency, (2) cleaning frequency, (3) number of passes, (4) equipment speed, (5) pavement conditions, (6) equipment type, and (7) public awareness [8, 9, 10].

Removal rates as reported in the literature vary considerably. In one study, the range was from 11 to 62% of the initial solids loading [11]. In another study, overall removal has been estimated at 33% of all pollutants on the street surface [9].

The relationship between concentration of pollutants found in urban stormwater and street sweeping frequency in one city is shown in Figure 21 [12]. The optimum interval can be determined by evaluating the trade-off of costs with effectiveness of sweeping.

Studies have also shown that the number of passes affects removal effectiveness [8, 7], as shown in Figure 22.

The effect of vehicle speed has been evaluated on residual debris. The optimum forward speed appears to be within the range of 5.6 to 8.0 km/h (3.5 to 5.0 mi/h) for efficient removal [7, 8].

The type of pavement affects both street cleaning efficiency and pollutant accumulation. While few data are available, in general, concrete pavements were found to be cleaner than asphalt streets. Pollutant loadings for asphalt surfaces have been estimated to be 7 to 20% higher than for other types of pavement [7].

The type of cleaning equipment also has an effect on the overall effectiveness of debris removal. Conventional sweepers are most efficient at removing larger contamination material, leaving behind the smaller fraction. Vacuum and air blast vehicles are capable of removing the smaller fractions. Vacuum equipment, however, rapidly loses its effectiveness when pavements are wet. This type of equipment has also experienced difficulties with clogged air hoses and filters due to clay-sized particles [6]. Water sprays can be used to remove street contaminants effectively; however, more frequent catchbasin and sewer cleaning may be required. The relative effectiveness of sweeping and flushing is shown in Figure 23.

Public awareness of, and participation in, street cleaning practices is essential for more efficient operations. Vehicles parked on streets during sweeping operations hamper efficiency and leave deposits untouched. Signs or flyers announcing sweeping schedules may result in more efficient operations [13]. One study [14] has concluded that 50% compliance with parking regulations yielded at best 25% of the curb swept. For 70 to 80% of the curb to be cleaned, the compliance with parking regulations must be maintained at 85% or higher.

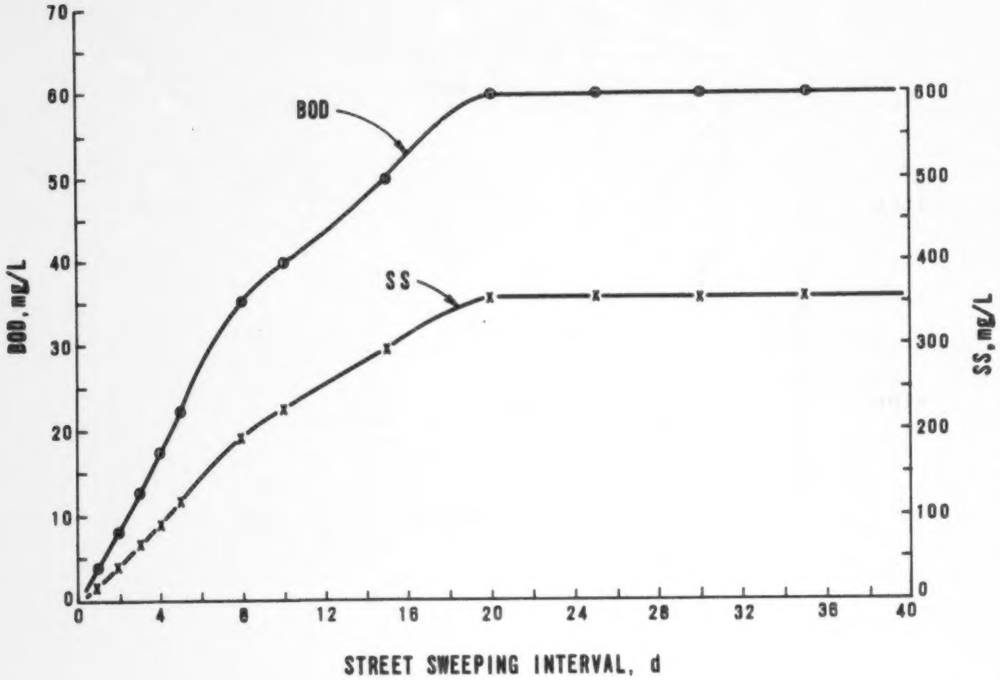


Figure 21. Effect of street sweeping frequency on mean BOD concentration in urban stormwater runoff, Des Moines, Iowa.

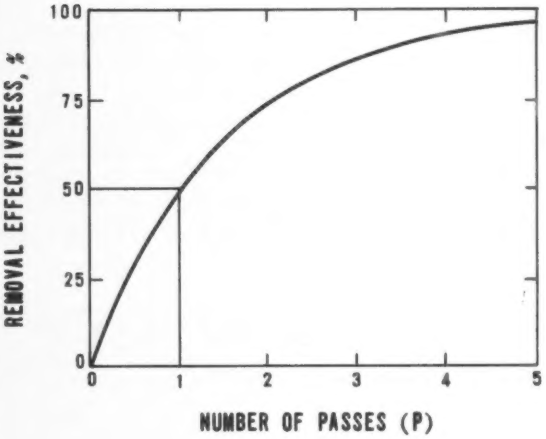


Figure 22. Street sweeping removal effectiveness with number of passes.

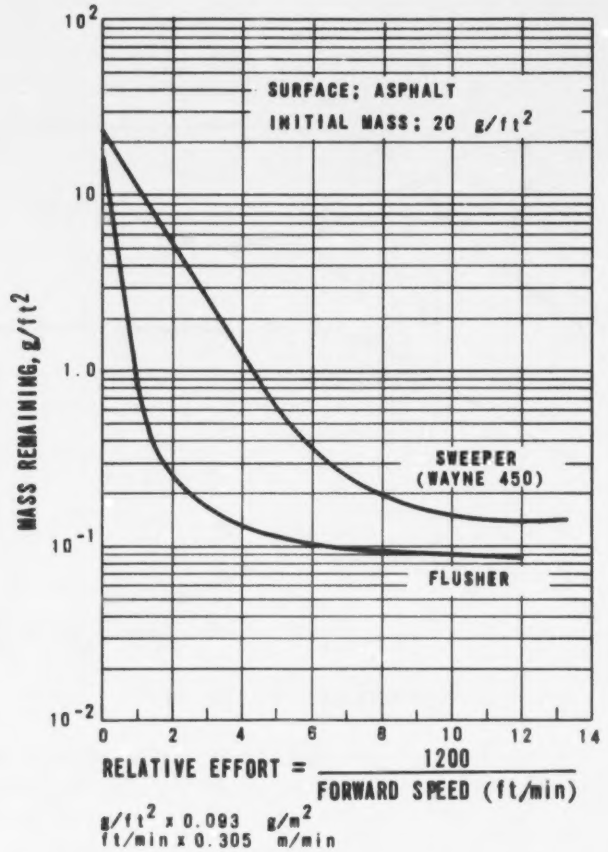


Figure 23. Comparison of cleaning performance of motorized street sweeping and motorized street flushing.

In terms of the reduced capabilities of the street sweeper as a result of illegal parking, if the assumption is made that a sweeper could remove 50% of the total solids, then 50% compliance would result in an overall efficiency of (0.50 x 0.25 x 100 =) 12.5%. Eighty-five percent compliance would cause a 37.5% reduction in total solids. An additional problem is the removal of abandoned cars, which impede sweeper activities. In one section of Manhattan, cars were abandoned at the rate of approximately 150 per month. Although minor in comparison to the more frequent case of illegal parking, it still acts as an impediment to the technique.

Costs of street cleaning have been reported to range from \$1.97 to \$7.61/curb km (\$3.17 to \$12.24/curb mi) swept (ENR 2000). The wide variation in these costs was attributed to differences in labor rates, unionization, and equipment costs [7].

Street Maintenance--

Pavement conditions have been found to have an effect on the amount of pollutants found on the street [7]. A curve relating pavement condition and solids loading for different land uses [9], is shown in Figure 24. In one report it was noted that as vehicles travel over rougher streets, more particulate matter is shaken off. A large portion of the solids also come from cracks in the pavement itself [13]. In terms of stormwater pollution, the optimum level of street maintenance could be determined by comparing costs of maintenance with the accumulation of pollutants.

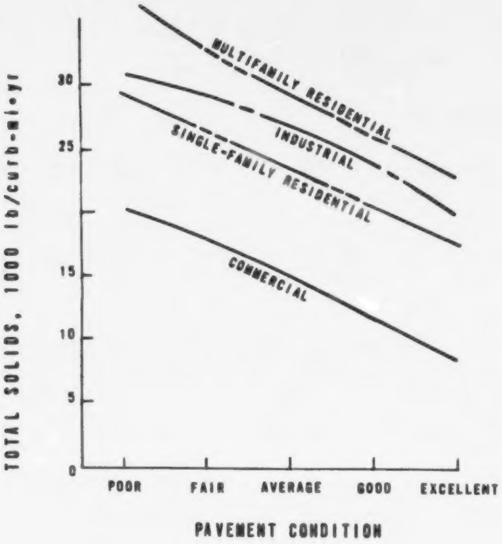


Figure 24. Effects of pavement condition and solids loading.

Management of Highway Deicing--

Effective management of highway deicing practices can lessen receiving water impacts associated with chlorides, sodium, and suspended solids, often without a substantial increase in costs.

Adverse environmental effects have been reported from the use of current deicing practices. Increases in chloride loads of 40 to 50% were experienced during the winter months, and chloride concentrations in the range of 1000 to 10 000 mg/L were found in receiving waters [15, 16, 17]. Sand contributes significant portions of suspended solids and causes maintenance problems with respect to catchbasins, and clogged storm and combined sewers [18]. Use of chemical additives such as cyanide, phosphate, and chromium can result in polluted snowmelt. Chromium concentrations of 3.9 mg/L have been reported [17].

Recommended alternatives modifying current deicing practices include: (1) judicious application of salt and abrasives; (2) reduced application rates using sodium and calcium salt premixes: rates of 42 to 113 kg/lane km (150 to 400 lb/lane mi) have been recommended [19]; (3) using better spreading and metering equipment and calibrating application rates; (4) prohibiting use of chemical additives; (5) providing improved salt storage areas; and

(6) educating the public and operators about the effects of deicing technology and the best management practices [16-19].

Cost associated with salting of roadways, both direct and indirect, were estimated on an annual basis for the snowbelt states [20]. A total annual cost of $3 billion was reported, of which only $200 million was associated with salt purchase and application. Other costs in the total estimate included (1) water supplies and health, $150 million; (2) vegetation, $50 million; (3) highway structures, $500 million; (4) corrosion damage, $2 billion; and (5) utilities, $10 million.

Collection System Maintenance

The major objective of maintenance of storm or combined sewer systems is to provide for maximum transmission of flows to treatment and disposal, while minimizing overflows, bypasses, and local flooding conditions. This objective can be achieved by maintaining the facilities within the system at their peak carrying capacity.

The significance of collection system maintenance as a best management practice is that when properly applied, extraneous solids and debris are removed in a controlled manner and thus do not accumulate as pollutant sources to be flushed into receiving waters under storm conditions.

The basic part of a maintenance program is regular inspection of the system. Specific tasks include: (1) catchbasin maintenance; (2) cleaning (both deposits and root infestations) and flushing of pipes; (3) removal of excess shrubbery, debris, and silt from flood control channels and ditches; and (4) control of inflow and infiltration sources.

Catchbasin Maintenance--

A catchbasin is defined as a chamber or well, usually built at the curbline of a street, for the admission of surface water to a sewer or subdrain, which includes at its base a sediment sump designed to retain grit and detritus below the point of overflow. The distinction is made between catchbasins as devices which intentionally trap sediment and storm inlets which do not have sumps and as a result should not retain sediment.

Historically, the role of catchbasins was to minimize sewer clogging by trapping coarse debris and to reduce odor emanations from low-velocity sewers by providing a water seal. With improvements in street surfacing and attention to design for self-cleaning velocity in sewers, their benefits were considered marginal as far back as 1900 [21]. Despite the purported reduced need, catchbasins are still widely used.

The area drained by a single basin is highly variable, averaging 0.63 ha (1.56 acres) in states with heavy snowfall and 0.88 ha (2.17 acres) in all states [22].

Catchbasins receive pollutants through the washoff of street surfaces and deliberate dumpings of crankcase drainings, leaves, grass clippings, pet feces, etc. Survey results from samplings from several basins [18, 23] show a wide range of potential pollutant loadings in the retained liquid. For example, samples from 47 basins in San Francisco showed COD variations from 153 to 37 700 mg/L, a BOD range of 5 to 1500 mg/L, and total nitrogen, 0.5 to 18.2 mg/L. Normalizing the data by casting out the extremes and averaging, the characteristics reduce to: COD, 6400 mg/L· BOD, 110 mg/L· total nitrogen, 8 mg/L· and total phosphorus, less than 0.2 mg/L.

Using these averages, the approximate BOD_5 pollutant load held in a basin computes to 0.08 kg (0.18 lb), or the equivalent waste discharged by one person in one day. A rainfall intensity of 0.025 to 0.050 cm/h (0.01 to 0.02 in./h) lasting 4 hours is sufficient to displace 90% of the liquid contents of a catchbasin [24]. Thus, for a city the size of San Francisco, even a minor storm may discharge the wastewater equivalent of 25 000 people through the purging of catchbasins. If not intercepted, this is equivalent to reducing the net dry-weather plant effectiveness by 3% on the day of the storm.

Countering this negative impact is the removal of pollutants associated with the solids retained in the basin. Sartor and Boyd [7] have identified pollutants in street surface contaminants associated by particle size in the dry state. Using hydraulic modeling analyses (approximate model to prototype scale ratio of 1 to 3), Lager, Smith, and Tchobanoglous [25] have reported catchbasin removal efficiencies as a function of basin geometry, flowrate, influent solids gradation, and accumulated solids from prior events. From these data, preferred design criteria were recommended for new construction, as shown in Figure 25. The performance of the recommended basin under a graded simulant load is shown in Figure 26. The impact of accumulated sediment in the basin did not materially affect removal efficiencies until 50% of the sump had filled. Under further loads, the removals dropped rapidly. Negative efficiencies were experienced before 60% of the sump had filled. Total accumulations by particle size at the point of breakthrough are shown in Table 64. In the estimates of the BOD_5 pollutant load, it is assumed that 50% of the street contaminants remain with the solids and that the balance goes into solution.

If only half of the available street contaminants in an urban area reach a catchbasin in a typical storm, approximately 0.24 kg (0.53 lb) of BOD_5 could be retained [25]. This beneficial retention is approximately three times the adverse purged pollution computed above, provided that the basin is well designed and maintained.

Cleaning methods fall into four main categories: hand cleaning, bucket cleaning, eductor cleaning, and vacuum cleaning. Comparison of APWA survey data [22, 23] from 1959 and 1973 shows that, on a national basis, the median cleaning frequency has decreased from twice per year in the earlier survey to once per year at present. This trend is obviously detrimental from a water quality aspect and illustrates that many problems associated with catchbasins may be traced to inadequate maintenance.

In general, catchbasins should be used only where there is a solids transporting deficiency in the downstream collection sewers and drains or at specific sites where available surface solids are unusually abundant (such

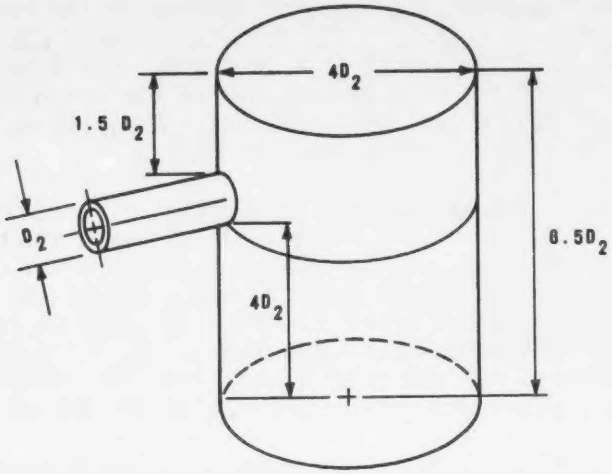
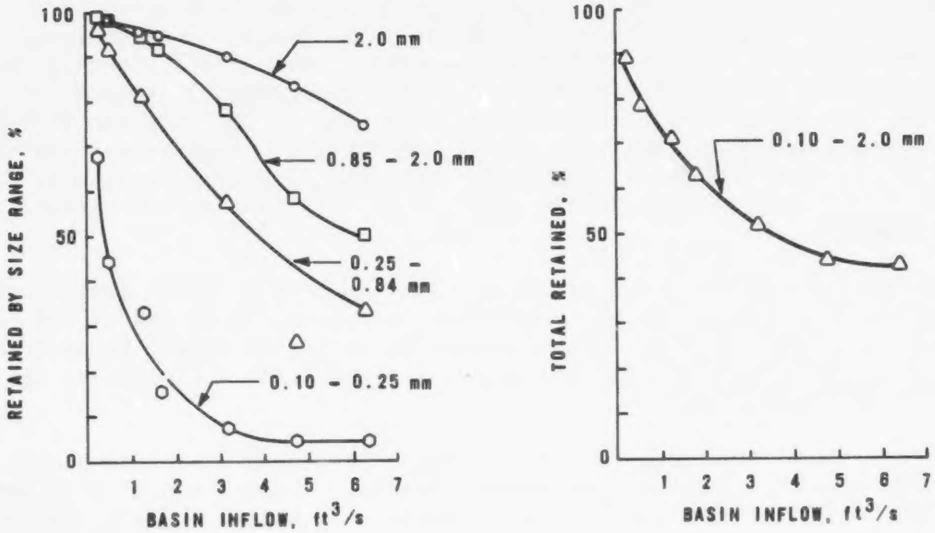

4D$_2$
1.5 D$_2$
D$_2$
8.5D$_2$
4D$_2$

Figure 25. Recommended design.



ft^3/s x 0.028=m^3/s

Figure 26. Solids removal efficiencies.

as beach areas, construction sites, unstable embankments, etc.). The advantages to be considered in the conversion of existing catchbasins to inlets are (1) a direct reduction in the "first flush" pollutant load, (2) a reduction in required maintenance, and (3) the opportunity to reallocate the conserved labor. Where catchbasins are required, the recommended cleaning frequency should be adjusted to limit the sediment buildup to 40 to 50% of the sump capacity [25], but in no case less than once per year.

TABLE 64. EXPERIMENTAL EFFECTIVENESS OF CATCHBASINS

Particle size, mm	% of applied solids retained	Equivalent BOD_5 removed, %[a]
0.10	0.0	0.0
0.10-0.25	10.0[b]	3.3
0.26-0.84	47.8	7.6
0.85-2.0	75.4	3.8
2.0	90.1	3.1
Total	57.5	17.8

a. Percent of total level washed from streets.

b. Estimated value.

Sewer Cleaning and Flushing--

Sewer cleaning involves routine inspection of the sewer system. Any lines found to be plugged or restricted should be scheduled for cleaning and cleaned. The major causes of restrictions in large-diameter sewers are siltation and accumulation of large debris like shopping carts and tree branches. In small-diameter sewers, siltation and penetration of tree roots are the major problems. Beneficial aspects include reduced local flooding and reduced emergency-type repairs as well as pollutant loading reduction. Increased carrying capacities and reduced blockages in interceptor/regulator works may directly reduce overflows.

Many variations of sewer cleaning equipment have been used, covering a variety of hydraulic, mechanical, manual, and combination devices. The basic concept involved in cleaning of sewers is that a cleaning tool is pushed or pulled through the sewer to remove an obstruction or it causes the obstruction to be resuspended in the flow and carried out of the system. The cost of sewer cleaning is highly variable and dependent on the particular type of obstruction encountered as well as the physical configuration of the system. Experienced costs for cleaning small and medium diameter circular sewers are summarized in Table 65.

TABLE 65. CLEANING COSTS FOR CIRCULAR SEWERS [26]

Pipe size, in.	Cost, $/ft
6-10	0.30-1.30
12-18	0.35-2.25
21-24	0.70-4.25
30-36	1.15-6.80

in. x 2.54 = cm
$/ft x 3.28 = $/m

The cleaning of large sewers and interceptors involves some unique problems because sludge blankets several feet deep can accumulate. The removal of these sludges can be pursued in several ways. In Boston, manhole modifications to existing manholes were made to provide access to 3970 m (13 000 ft) of a 1.53 m (5 ft) diameter interceptor containing an estimated 3030 m^3 (4000 yd^3) of sludge. A 0.38 to 0.77 m^3 (0.5 to 1 yd^3) cresent-shaped bucket was used with winches to drag the sludge from the sewer. Removal of the sludge costs approximately \$46/m^3 (\$35/yd^3). The total cost of the project was estimated to be $37.70/m ($11.50/ft) of sewer [27].

Sewer Flushing-- Flushing of sewers on a regular basis can ensure the continuing capability of sewer laterals and interceptors to carry their design capacity as well as alleviate solids buildup reducing solids to overflow.

Sewer flushing can be particularly beneficial on sewers with very flat slopes (i.e., too flat for average flows to maintain sand and grit particles--with their associated contaminants--in suspension at all times). If a modestly large quantity of water is discharged through these flat sewers periodically, small accumulations of solids can be washed from the system. This cleaning technique is generally effective only on freshly deposited solids.

Internal automatic flushing devices have been developed for sewer systems. An inflatable bag is used to stop flow in upstream reaches until a volume capable of generating a flushing wave is accumulated. When the correct volume is reached, the bag is deflated with the assistance of a vacuum pump releasing impounded water and cleaning the segment of sewer [28].

The flushing wave will be attenuated by wall friction and other internal pipe irregularities and has limited usefulness. It is estimated that approximately 370 m (1200 ft) of small and medium diameter sewer could be flushed by a single flushing station.

For 46 to 61 cm (18 to 24 in.) pipes, an automatic flushing station capable of being installed in an existing manhole would cost approximately $6000. This station would require routine inspection (once a month). It would have an estimated annual operating cost of $500 and would consume approximately $50 worth of power annually.

Drainage Channel Maintenance-- Maintenance of flood control channels can cover a wide range of cleaning tasks. Debris to be removed ranges from trash, garbage, and yard trimmings to used tires and shopping carts. Currently, very little cost data are available on the maintenance of flood control channels. A limited survey of West Coast flood control districts indicates that the cost for maintenance of flood control facilities range from $2.50 to $5.75/linear m ($0.75 to $1.75/linear ft) foot) of facility. This cost would be affected by the size of channel, the type of channel lining, and the access to the channel. Other facility maintenance costs are shown in Table 66.

TABLE 66. OPERATION AND MAINTENANCE COSTS FOR FLOOD CONTROL FACILITIES

Item	Factor-yearly cost
Concrete structures	0.1% of first cost
Gates and steel structures	1.5% of first cost
Levees	$960 + $16/ft height·mi
Riprap slopes	$70/ft height·mi
Unlined slopes	$144/ft height·mi
Channel	$320/mi
Roads and berms	$400/mi
Interior drainage facilities	2% of first cost
Undeveloped riverbanks	150% of developed bank cost

$/ft height·mi x 2.038 = $/m height·km
$/mi x 0.622 = $/km

Inflow and Infiltration--

The entering of extraneous flows into a sewer can be generally categorized as either inflow or infiltration. Inflow usually occurs from surface runoff via roof connections, cross connections between sanitary and storm sewers; yard drains, or flooding of manhole covers. Infiltration usually occurs by water seeping into the pipe or manholes from leaky joints, crushed or collapsed pipe segments, leaky lateral connections, or other pipe failures. By reducing effective collection system and treatment plant capacities, extraneous flows may result in unnecessary pollution.

The location and assessment of inflow-infiltration sources have been discussed at length in recent literature [26, 29]. The cost for an evaluation survey to determine the magnitude of inflow/infiltration ranges from $3.60 to $7.70/m ($1.10 to $2.35/ft) of sewer (1974 dollars).

Some of the most common sources of inflow are summarized in Table 67. Estimates are given for the flow contributed from each source and the cost of eliminating that source are also given.

TABLE 67. REHABILITATION COST ESTIMATES FOR INFLOW ELIMINATION

Inflow source	Flowrate, gal/min	Rehabilitation cost (ENR 2000), $
Leakage around manhole covers	10-20	50-75
Holes in manhole covers	50-100	100-125
Foundation drains	10	300-1200
Roof leaders	10	50-75
Cross connection	250-450	100-500
Catchbasin	300	3000-5000
Ditch or storm sewer--infiltration sanitary sewer (per manhole reach)	60-80	500-2500
Area drains	50-200	50-350

gal/min x 0.0631 = L/s

Infiltration is usually more difficult to locate, and more expensive to cure. The worst possible case would require replacement of a segment of sewer between two manholes (commonly called a reach). If only minor infiltration exists, a small-diameter sewer can be grouted using pressurized pumped sealants. Typically, grouting a manhole reach with up to three infiltration sources costs approximately $600. Detailed costs for the control of inflow/infiltration sources are presented in recent literature [26].

Onsite Storage of Runoff

The objective of onsite storage of runoff is either to prevent storm flow from reaching the drainage system or to change the timing of the runoff by controlling the release rate. Retention is the term for total containment, and detention is the common term for delaying and controlling the release rate to smooth out the peak flows.

Use of onsite storage allows a natural drainage system or a less extensive, less costly manmade system to be used.

Pollution Control--

The ways in which stormwater pollution is reduced by onsite storage are as follows:

- There is no surface water pollution loading from retention facilities. (However, the effect of infiltrating water on groundwater should be analyzed.)

- Delaying runoff and stretching out the pollutant loading over a period of time enhances the ability of the stream to assimilate pollutants.

- Planned onsite storage can be used to keep rainwater from running off urban surfaces that are potential sources of pollution, such as urban streets.

- The decreased velocity of storm runoff caused by detaining peak flows will result in less channel erosion in natural streams and earthen conduits. The lower velocities also mitigate the impact on organisms living in the stream.

- If peak flowrates of stormwater through combined sewers can be controlled, the overflows of heavily polluted combined sewage can be decreased or eliminated.

- Detention of stormwater in a pond for any period of time will result in some settling and thus may decrease the particulate loading of the outlet water. Some biological stabilization may also occur.

The precipitation/infiltration process is the most important method of replenishing the groundwater reservoirs that serve as potable water supplies for many areas of the country. The decreased infiltration and increased water demand caused by urbanization will stress groundwater supplies unless recharge areas are set aside as basins develop. Although large-scale urban stormwater recharge programs have not been implemented because of potential groundwater pollution, onsite retention and recharge has been developed for small watersheds. Retention basins are usually variable-depth ponds designed with no outlet or only a bypass for exceptionally high flow conditions.

Retention is also practiced as controlled onsite storage where groundwater recharge is not important. In a typical example, the California Division of Highways has built retention basins to dispose of highway runoff in the San Joaquin Valley. These basins were developed from 0.4 to 2.4 ha (1 to 6 acre) depressions that had originally been excavated for embankment material. Infiltration capacity is sometimes improved by excavating 1.8 to 3.1 m (6 to 10 ft) deep trenches or vertical drains and backfilling with porous material. Maintenance is minimized by providing low-velocity channels ahead of the basins to help settle suspended particles. The areas are scarified once a year to decrease the surface clogging effects of organic solids.

The alternative to retention is to construct sewers to carry the runoff to acceptable receiving waters. Therefore, the economic advantages depend on the length of sewer that would be required. Additional advantages of the ponds include total containment of the highway runoff pollutants and the recreational asset to local cities that can landscape the basins as additional parkland.

Detention-- In its simplest form, detention means capturing stormwater and controlling the release rate to decrease downstream peak flowrates. Onsite detention uses simple ponding techniques on open areas where stormwater can be accumulated without damage or interference with essential activities. The design essentials include a contained area that allows the stormwater to pond and a release structure to control the rate at which the runoff is allowed into the drainage system. The release structure is usually a simple construction, such as a small-diameter pipe draining a basin or an orifice plate placed at a sewer inlet. The capacity of the pipe or orifice limits the flowrate to a level acceptable to the downstream system. Where the depth of ponding has to be limited, the release structure will have an automatic overflow to prevent excessive ponding.

Surface ponding is the most common form of detention being used by developers. In most cases the facilities are carefully planned so that the ponding area is a dual-use facility that enhances the value of the site. Variable level ponds have a permanent water level during dry weather and increased holding capacity during storm conditions. The permanent lakes have aesthetic and recreational appeal which increases lot values. Basins that are dry between storms are often designed to be used as baseball fields, tennis courts, and general open space. Parking lots can be made to serve as low-depth storage ponds by sloping the sidesand constructing drain outlets. Side slopes are restricted to about 4 percent for traction in the winter, and the pond depth is limited by the need for people to reach their vehicles. Obviously, a truck terminal lot can be allowed to pond to a greater depth than a supermarket lot. The economic aspect of surface ponding is derived from the savings over a conventional sewer project. Several surface ponding sites are listed in Table 68. A cost comparison is also made between a drainage system using surface ponds to decrease peak flows and conventional storm sewer construction. Greater benefits are obtained if the pollution control aspect is considered.

TABLE 68. SURFACE PONDING [30]

		Cost estimate, $	
Site	Description	With surface ponding	Without surface ponding
Earth City, Missouri	A planned community including permanent recreational lakes with additional capacity for storm flow	2 000 000	5 000 000
Consolidated Freightways, St. Louis, Missouri	A trucking terminal using its parking lot to detain storm flows	115 000	150 000
Ft. Campbell, Kentucky	A military installation using ponds to decrease the required drainage pipe sizes	2 000 000	3 370 000
Indian Lakes Estates, Bloomington, Illinois	A residential development using ponds and an existing small diameter drain	200 000	600 000

Two variations of detention that have proven successful for metropolitan application are ponding on plaza areas and ponding on roof tops. Both techniques have been pioneered at the Skyline Urban Renewal Project in Denver, Colorado [31]. The basic approach is the same for other forms of detention. The outlet from the ponding area must be constructed to allow runoff to accumulate during peak storm conditions. The depth that can accumulate on plazas must be limited to approximately 1.9 cm (0.75 in.) because of pedestrians, but it is possible to design plazas so that portions can be flooded without inconvenience. A depressed plaza section in Denver is shown in Figure 27. Roof tops in metropolitan areas provide an excellent opportunity for stormwater detention. Most are flat, watertight, and structurally designed to take loads greater than that of ponded stormwater. It adds very little to the cost of a new building to ensure structural conditions for ponding. Detention is controlled by a simple drain ring set around the roof drains. As the roof begins to pond, flow is controlled by orifices in the ring; extreme flows overflow the ring to prevent structural damage to the roof.

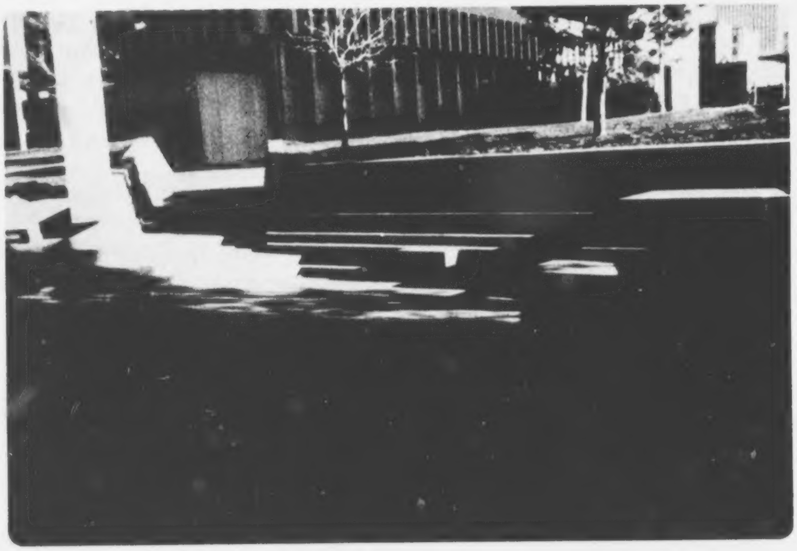


Figure 27. Depressed plaza ponding, Denver, Colorado.

Design Considerations--

The acceptability of onsite storage as a pollution control alternative depends on the mitigation of apparent adverse factors, including the safety hazard to children, maintenance difficulties, mosquito breeding, algae growth, the land area required, possible poor appearance of dry ponds, and the responsibilities of ownership.

Safety-- The safety features depend on the secondary use of the facility. Obviously, a dual-purpose recreational lake cannot be fenced to prevent access. Typical safety features include shallow bank slopes and outlet guards.

Maintenance-- Debris removal, care for the landscaping, and maintenance of the outlet structure are all part of the routine operation of a detention facility.

Mosquito breeding and algae growth-- Both mosquito and algae problems can be eliminated from dry basins by ensuring that the areas dry out completely between uses. For permanent ponds, these problems are more difficult to control. Mosquito breeding can be upset by controlling grass at the shoreline, varying the water depth every few days, or stocking the ponds with larvae-eating fish.

Land area required-- The best way to overcome objections to land set aside as a detention pond is to recognize that the area can be an asset as open space. Housing near greenbelts and pond areas usually has a higher market value if the open space is aesthetically designed.

Poor appearance of dry ponds-- Detention ponds are most presentable when a grass cover is kept on the basin slopes and floor. Grasses can be grown that will withstand periodic flooding. If retention basins contain water for long periods of time or need to be vegetation-free for better infiltration, appearance objections may be overcome by sight barriers such as trees.

Responsibility of ownership-- In most cases the responsibilities of operation and ownership should be assumed by a public agency. The equipment, manpower, and expertise required for operation and maintenance is beyond the abilities of homeowner associations and developers.

LEGISLATION

Special legislation is necessary to implement many best management practices effectively. Laws, ordinances, and agreements will simplify the process of draining upland areas with a minimum of flood damage and pollution. The alternative, civil suits and tort law, becomes almost unworkable when thousands of property owners are involved. The simplest form of legislation, and the form enacted in most urban areas, provides for a public works authority to build and maintain a drainage system to transport runoff to a major receiving water stream. The authority is funded or empowered to raise money and allowed to acquire property or easements for the system.

In most cases, the flood control or drainage authority succeeds in meeting its primary objective, quick removal of stormwater, by channeling runoff into large concrete conduits and discharging at a point downstream from the newly urbanized area. Some major difficulties can develop from the implementation of the typical runoff system.

1. The combination of a generally impervious urban area and a system designed to remove runoff as quickly as possible will increase flood peaks in the receiving stream. This increase may be disastrous for downstream residents.

2. Urban runoff has the pollutional characteristics of a weak sanitary sewage and consequently there is a growing realization that some type of treatment should be required before discharge into sensitive receiving waters. Since many collection and treatment components are necessarily sized on the basis of the peak rate of flow, it would be cost effective to decrease both runoff peaks and volumes.

3. The common growth pattern for an urban area consists of an older city on the banks of a receiving stream with suburban areas developing on the perimeter of the city. These newer suburban towns must often drain through the original city to the stream. Aside from jurisdictional problems, the increased flow will tax the capacity of the city's system in highly developed areas where construction to increase capacity is difficult and expensive.

One solution to these problems is drainage basin legislation requiring that hydrologic changes be kept at a minimum during development. If the rainfall and runoff pollutants are contained at the source, the downstream problems are mitigated. Legislation to accomplish the containment is more complex and controversial than common drainage laws. At a minimum it will require that a percentage of the land be left undeveloped and dedicated to infiltration or detention basins. The legislation may be very controversial if it severely restricts the ways in which private land can be developed.

General Concepts

The development of a runoff policy requires that a legislative authority representing the drainage basin (usually a county or state) study the problems of the basin, formulate objectives, and outline methodology for meeting the objectives. The actual engineering involved in detailing the methodology is usually developed in a manual of regulations by the public works authority designated to enact or enforce the legislation.

The problems that must be defined will vary by basin and receiving water but generally fall into the categories of flood protection, pollution abatement, erosion control, and groundwater protection. The objectives of the legislation will include some of the following points:

- To protect the public health, safety, and welfare
- To define responsibility for all aspects of the problem, i.e., will solutions be developed by cities, developers, or private landowners?
- To authorize administrative and/or public works departments to implement the legislation
- To implement the most equitable and cost-effective solution to the problem
- To protect the receiving water
- To conserve stormwater for beneficial use

- To control development in the flood plain
- To provide a basis for future development by considering areas where growth should be controlled or encouraged

The objectives have to deal with future conditions as well as the present and in fact legislation is more successful in preventing future problems than in solving existing ones.

The basic methodology is the containment of all or part of the runoff and pollutants at or near the source. The options for accomplishing this will be illustrated by examples of programs in several areas. The choices may include:

- Regulations requiring that the rate or volume of runoff after development be the same as predevelopment levels
- A program by a municipal authority to build upstream detention or retention facilities
- Regulations prohibiting construction in natural ponding areas or floodways
- A system for runoff control taxes that are prorated according to the amount of runoff generated from the property
- Erosion and sediment control ordinances designed to prevent soil loss, especially during construction activities
- Anti-littering and discharge ordinances that prohibit the use of channels and stormwater systems to dispose of refuse, motor oil, and other foreign material

The methodology should be presented in the legislation to give a clear mandate to the implementing authority. However, it is not necessary to give specific design regulations for engineering solutions required. For example, it is preferable to require that runoff not exceed historic rates for the 10 year storm rather than mandate a certain size retention basin. The former case will allow the landowner or municipality to develop the best engineering solution for the individual site.

The legislation can take any of several forms depending on the authority and objectives of the legislative body. Examples would include municipal ordinances, flood control ordinances, building codes, zoning plans, subdivision regulations, sewer and drainage fee assessments, greenway or open space plans, and pollution control ordinances.

Example Programs

A summary of ten innovative programs as reported in the literature [30, 32] is presented in Table 69.

TABLE 69. SUMMARY OF LEGISLATIVE STORMWATER MANAGEMENT PROGRAMS

Location	Description of legislation
Denver Urban Renewal Authority	Requires private developers to pond rainfall on rooftops and in plazas of all new and renovated construction. The design criteria for plazas require a runoff rate of 1 in./hr and a water depth of 0.75 in. during the 10 year rain. The values for rooftops are 0.5 in./hr and a depth of 1 in. for the 10 year storm or 3 in. during a 100 year rain.
Naperville, Illinois	Plumbing, sewer, and water ordinance requiring that runoff release rate be regulated by the safe capacity of the receiving water, but no more than 0.15 in./h. Storage must be designed for the 100 year storm. The ordinance is applicable to all new subdivisions and compliance is required for approval of development permits.
Joliet, Illinois	Ordinance similar to that of Naperville. Requires runoff to meet a variety of criteria: (1) runoff rate shall not exceed historic values, (2) allowable runoff rates are prorated on the basis of stream capacity, and (3) runoff rate shall not exceed that of 2 year storm with a runoff coefficient of 0.3 unless facilities can handle the flow. The ordinance is enforced for 10 acre residential areas and 5 acre nonresidential developments through the issuance of building permits.
Albuquerque Metropolitan Arroyo Flood Control Authority	Requires stormwater detention for all new developments such that downstream drainage facility capacity is not exceeded or the rate of runoff does not exceed the natural rate of flow. Compliance is required for building permits and subdivision plat approval. In addition, a land use not in compliance can be sued as a public nuisance.
Arvada, Colorado	Requires detention for runoff greater than predevelopment rates for new construction. If a developer chooses not to provide the detention he is assessed a one time fee that reflects the cost the city will pay to develop a drainage system. If detention is provided, no fee is assessed.
Boulder, Colorado	Monthly drainage fee that is assessed against all property in the city on the basis of surface area and runoff coefficient. Efforts to retain runoff onsite will result in lower monthly charges.
Metropolitan Sanitary District of Greater Chicago	Requires provision for stormwater retention before granting sewer connection permits to new developments. The maximum release rate is computed by the Rational Formula with a 3 year rain and a coefficient of 0.15. Storage must be designed for the 100 year storm.
Montgomery County, Maryland	The State of Maryland has classified sediment as a pollutant under its Water Pollution Control Act and Montgomery County's program is an example of the result. The recommendations of the SCS on erosion control must be met to obtain clearing and grading permits in the county. Detention ponds are part of the requirements for approval.
Fairfax County, Virginia	The county has a history of runoff control similar to that of Montgomery County. Erosion and sediment control has been mandated during construction since the late 1960s. Temporary detention ponds were used at most sites and permanent detention must be evaluated for all new developers.
Springfield, Illinois	Sewer ordinance for combined sewer areas that has decreased runoff by a successful campaign to disconnect sewer downspots from the sewer system.

in. x 2.54 = cm
acre x 0.405 = ha

Model Ordinance Outline

A model ordinance has been developed in reference [32]. The following outline covers many of the recommended points.

1. Scope. The ordinance is referenced to existing legislation to prevent overlap or conflict.

2. Definitions. Engineering terms and concepts used in the ordinance should be clarified.

3. Objectives. This section is used to give direction to ordinance and to help the citizenry and courts understand the purposes of the law. Several objectives were listed earlier.

4. Floodplain Regulation. Develop regulations for land use within the contours of the 100 year flood.

5. Hydrologic and Hydraulic Studies. Developers should submit runoff studies for a proposed project. The reports would contain details of existing and projected runoff volumes and rates to serve as a basis for designing detention facilities and measuring potential impacts on downstream systems.

6. Improvements Required. Depending on the objectives of the ordinance, improvements may be required to meet runoff standards. Detention facilities could be required and a maximum release rate specified. This is the most important part of the ordinance as it is where the chosen methodology is developed.

SUMMARY

Nonstructural and low structurally intensive alternatives, termed best management practices (BMPs), offer considerable promise as the first line of action to control urban runoff pollution. By treating the problem at its source, or through appropriate legislation curtailing its opportunity to develop, multiple benefits can be derived. These include lower cost, earlier results, and an improved and cleaner neighborhood environment.

The greatest difficulty faced by BMPs is that the action-impact relationship are almost totally unquantified. It is clear that onsite storage, for example, can be closely related to reduced downstream conduit requirements but the net water quality benefits are far less defined. Similarly, cleaner streets and neighborhoods and enforced legislation will eradicate gross pollution sources but to what limit should they be applied and who will bear the cost? The final answers of cost effectiveness will not be found short of trial implementation. Key demonstration projects in this regard, both in the early planning stages, are expected to be implemented in Belleview, Washington (4 years), and Orlando, Florida (2 years) [33].

The alternatives, or preferably supplements to BMP, are discussed in the next section.

SECTION 7

UNIT PROCESSES

Many treatment alternatives are available to planners and designers to control stormwater pollution; they have been demonstrated either on an individual basis or as dual use facilities in conjunction with dry-weather treatment facilities. The stormwater treatment alternatives presented in this section include storage, physical treatment, biological treatment, land treatment, and disinfection. The alternatives are discussed on a unit process basis; however, individual processes or combinations of processes may be implemented on various scales to produce the required degree of treatment.

It has been concluded that some form of storage or flow equalization must be considered in implementing these stormwater treatment options to reduce, in size, number, and costs, the treatment facilities required [1].

Actual operational data for most demonstration and prototype stormwater facilities are limited, but it is emphasized that the receiving water condition and/or degree of receiving water improvement be evaluated on a cost-effectiveness approach. Models and modeling techniques have been instrumental in this regard.

Master planning approaches using storm and combined sewer treatment processes are discussed in Section 8, Applications, for several case history sites.

STORAGE

Because of the high volume and variability associated with storm and combined sewer overflows, storage is considered a necessary control alternative. Storage facilities are frequently used to attenuate peak flows associated with these discharges, reducing in magnitude and size of facilities required for further treatment. Storage, however, with the resulting sedimentation that occurs due to increased detention times, can also be considered a treatment process. Many such facilities are designed to operate as sedimentation basins as well as storage tanks for flows that exceed the storage capacity. Characteristics of sedimentation systems are described under Physical Treatment Alternatives.

Storage facilities may be analyzed and designed by various rational methods [2], however, recent studies offer a cost-effectiveness approach for sizing storage facilities coupled with secondary treatment for various pollutant removal constraints [3, 4]. This approach provides a first-cut methodology for comparing alternative costs at different levels of treatment for different combinations of storage treatment processes.

The two types of storage facilities discussed include inline and offline storage. Source ponding and rate control were discussed previously in Section 6.

Inline Storage

Inline storage, the use of the unused volume in interceptors and trunk sewers to store runoff, is a particularly attractive option for controlling urban runoff. This alternative includes installation of effective regulators, level sensors, tide gates, rain gage networks, sewage and receiving water quality monitors, overflow detectors, and flowmeters and then applies computerized collection system control. Such systems have been developed and successfully implemented in Seattle, Minneapolis-St. Paul, and Detroit [2, 5-7].

The basic elements of a monitoring and control system may include all or combinations of the following: (1) remote sensors (rain gages, flow level and selected quality monitors--such as DO, TOC, SS, and/or pH probes, gate limit switches and position monitors); (2) signal transmission (leased telephone wires, pneumatic circuits); (3) display and logging (central computer, graphic panels, warning lights); (4) centralized control capability (control of system gates and/or pumps from a central location); and (5) in the case of fully automated control, a computer program that makes decisions and executes control options based on current monitoring data and memory instructions.

Descriptions of regulators commonly found in combined sewer systems along with installed construction and annual costs are found in the literature [2].

Inline Storage Effectiveness--

Several prototype inline storage facilities are currently in operation, showing satisfactory effectiveness in reducing total overflow volume and the number of overflow events. It has also been shown that as operators become more familiar with the system, the effectiveness of the system operation increases.

The Seattle computer controlled inline storage system, with an estimated maximum safe storage capacity of 67.5 ML (17.8 Mgal) in the trunklines and interceptors, has evolved through several control modes and is now operating under automatic control. The increased storage effectiveness as a result of increasing system control is shown in Figure 28 [5]. The regression lines represent data from 762 separate recorded overflow events from 341 out of 514 storm events during the 3-1/2 year demonstration period.

The Detroit Metro Water Department (DMWD) sewer monitoring and remote control system with an estimated 530 ML (140 Mgal) of controlled inline storage and an additional 568 ML (150 Mgal) of uncontrolled storage (storage that is not a result of the control system), operates in the supervisory control mode [7]. The system captured a total volume of wastewater amounting to 21 575 ML (5.7 billion gal) during the 18 month demonstration period. An estimated 3.2 million kg (7 million lb) of BOD and 5.9 million kg (13 million lb) of suspended solids were prevented from entering the Rouge and Detroit rivers. During the first 6 months of operation, the DMWD was able to completely contain an equivalent uniform depth of rainfall of 0.18 cm (0.07 in.) over Detroit's 363 km^2 (140 mi^2) area. Through operator experience and knowledge of the interceptor system, this was increased to 0.36 cm (0.14 in.) during the last 6 months of the demonstration period.

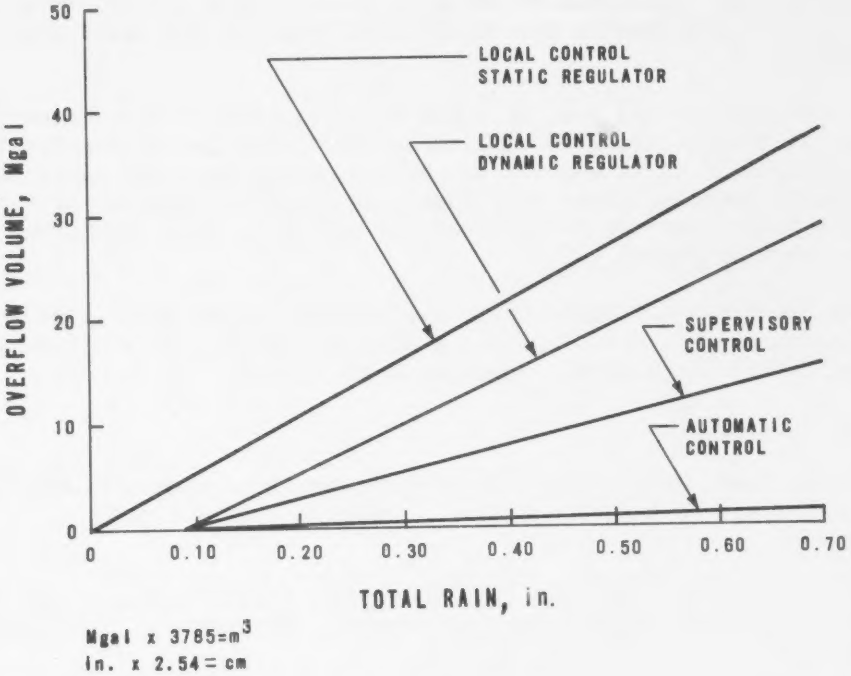


Figure 28. Inline storage effectiveness regression lines for each mode of control.

System Evaluation--

Management effectiveness of inline storage systems may be increased through equipment and hardware selection, implementation, early warning systems, and knowledge of the type of storm to be expected.

Seasonal storm patterns and types in Seattle have been identified as to their effect on the storage system [5]. The control system is used most often during the winter rainy season in which long duration rainfall events with moderate intensities are common. Heaviest loads are put on the computer system with these storms when interceptors are full or nearly full due to the long duration with areawide involvement.

Advanced warning systems have been credited for improved performance of inline storage systems. Surveillance and early attention to potential mechanical and control problems, early action and preparation for storm events, and logical use of all available system storage have contributed to Seattle's computerized system's success [5]. Radar displays of approaching storms gives Detroit system operators advanced warning of approaching storms and facilities pumpdown procedures to maximize storage capacity in the system [7].

Increased density of rain gages, level sensors, and overflow status monitors are recommended for increased system sensitivity. This is especially true when utilizing systems with limited storage capacity. Standardization of data collection, display, and computer hardware and software is recommended to prevent potential programming and interfacing problems and would greatly reduce costs [5-7].

Inline storage systems are also applicable for use during dry weather. System monitoring has enabled the DMWD to suspend pumping at the wastewater treatment plant for periods of up to 6 hours to perform maintenance and modifications without causing combined sewer overflows. In addition, monitoring has enabled DMWD to hold back flow from portions of the system to allow for sewer inspection and maintenance [7].

Recent literature has developed criteria, rationale, and guidelines for planners, managers, and designers concerning implementation of automation and control facilities for combined sewer systems [8-10].

Operational Problems--

Operational problems associated with inline storage control systems include computer programming and hardware design, and control equipment implementation.

To develop a functional computer control system, the following sequence of system design has been demonstrated to prevent reprogramming and redesign of the system [5].

1. Preparation of overall system design
2. Preparation of system programs
3. Preparation of applications programs

A system of debugging foreground programs on-line should also be provided. When system problems are encountered the following sequence of sources have been recommended:

1. Program bugs
2. Inadequate hardware documentation
3. Hardware malfunction
4. Hardware design deficiencies

Electrical noise has been the cause of many problems encountered in computer monitoring and control systems with telemetry or data transmission [5, 7].

This causes a loss of accuracy in the data requirements needed for system control.

Requirements for dependable service from control system equipment is paramount to efficient operation. Studies in Detroit show that although hydraulic operated regulators may be more maintenance free and faster acting, they may not provide the degree of safety to warrant their use. Hydraulic operators may tend to drift from their set position causing unwanted overflows, and are difficult to operate manually in case of failure. Electrically operated gates once positioned will not drift and can be manually overridden during power failure [7].

Costs of Inline Storage Systems--

Costs associated with inline storage systems are summarized in Table 70. Costs include regulator stations, central monitoring and control systems, and miscellaneous hardware.

TABLE 70. SUMMARY OF INLINE STORAGE COSTS[a]

Location	Storage capacity, Mgal	Drainage area, acres	Capital cost, $	Storage cost, $/gal	Cost per acre, $/acre	Annual operation and maintenance $/yr
Seattle, Washington [2, 5]						
Control and monitoring system	....	......	3 500 000	....	...	73 000
Automated regulator stations	....	......	3 900 000	....	...	219 200
	17.8	13 120	7 400 000	0.42	564	292 200
Minneapolis-St. Paul, Minnesota [2, 6]	NA	64 000	3 000 000	....	47	....
Detroit, Michigan [7]	140	89 600	2 810 000	0.02	31	....

NA = not available.

a. ENR 2000.

$/acre x 2.47 = $/ha
$/gal x 0.264 = $/L
Mgal x 3 785 = m^3

Offline Storage

Offline storage is used to attenuate storm flow peaks, reduce storm overflows, and capture the first flush, or provide treatment in the form of sedimentation when storage capacity is exceeded. Offline storage facilities may be located at overflow points or near dry-weather or wet-weather treatment facilities, depending on the type and function of the storage facility to be used. Offline storage may also be used for onsite storage of runoff, as described in Section 6.

Types of Offline Storage--

Offline storage facilities have been designed for flow containment to reduce in magnitude the peak flow entering downstream dry- or wet-weather treatment facilities, and for treatment by detention and sedimentation of stormwater before discharge to receiving waters. Simplified schematics of these operations are shown in Figures 29 and 30. Offline storage facilities used for sedimentation are discussed under Physical Treatment Alternatives.

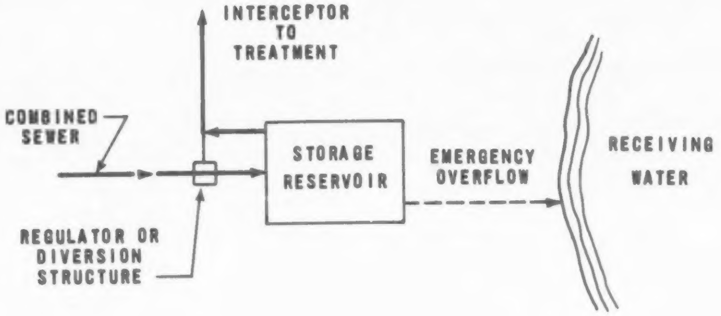

INTERCEPTOR TO TREATMENT
COMBINED SEWER
STORAGE RESERVOIR
EMERGENCY OVERFLOW
RECEIVING WATER
REGULATOR OR DIVERSION STRUCTURE

Figure 29. Flow schematic of storage used for containment.

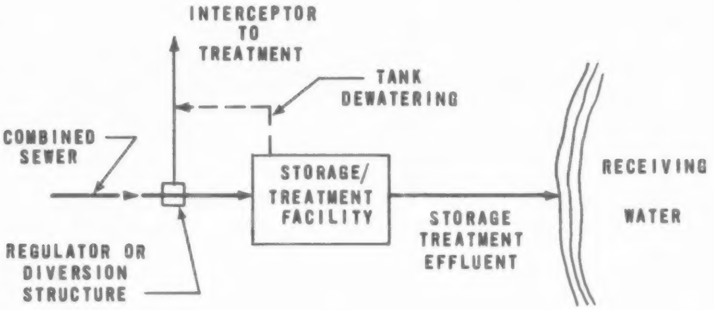

INTERCEPTOR TO TREATMENT
TANK DEWATERING
COMBINED SEWER
STORAGE/ TREATMENT FACILITY
RECEIVING WATER
STORAGE TREATMENT EFFLUENT
REGULATOR OR DIVERSION STRUCTURE

Figure 30. Flow schematic of storage/treatment facility.

Flow Containment--Flow containment facilities are usually constructed of earthen lagoons, reservoirs, or lined basins in low population density areas, less than 25 persons per hectare (10 persons per acre), where land costs are lower and availability is higher [3]. Storage facilities of this type usually contain in excess of 50% of the potential overflow volume per year.

Storage/Detention--Storage/detention facilities provide containment for small overflows and provide detention and thus primary treatment for overflows that exceed the storage capacity. They are usually characterized by covered concrete tanks, providing less than 50% overflow volume reduction and are constructed in high density areas, greater than 25 persons per hectare (10 persons per acre) because of land costs and land availabiilty [2, 3].

Storage System Characteristics--

Basic appurtenances common to storage facilities include flow diversion or regulation structures, coarse screening, storage overflow structures, and dewatering by pumping or gravity. In addition, storage/detention facilities which provide primary treatment may include all or combinations of the following:

- Fine screening of the influent
- Disinfection systems
- Fine screening or other treatment of the effluent
- Sludge/solids collection and removal

Sludge/solids collection and removal is perhaps one of the most important operations in the storage operation, as inadequate removal can generate volatile gas and cause mechanical malfunctions and odors. Typical collection equipment includes traveling bridge sludge scrapers and hydraulic dredges [2, 11, 12]; mechanical mixers, recirculation pumps, and compressed air for solids resuspension and removal [13-15]; automatic and manual flushing [16, 17]; and use of street sweepers in lined basins [18]. Use of automatic and mechanized methods of solids removal was shown to be more effective than manual washdown operations [12].

Design Criteria--

Storage facilities have been designed using concepts based on duration-frequency analysis of local rainfall events [2]. Storage selection and sizing should also incorporate receiving water conditions as part of the design criteria. Evaluation of the percent reduction of pollutants required to obtain the most cost-effective design must also be compatible with water quality goals.

Studies for Milwaukee have developed process curves for detention tanks, evaluating pollutant reduction and volumetric efficiency for several tank volumes. Suspended solids and BOD retention and percent of storm volume retained for both wet and dry year rainfalls are shown in Figure 31 [13]. The study also showed a decreasing efficiency per unit volume as tank size increases, as shown in Figure 32.

Offline Storage Effectiveness and Applications--

Offline storage facilities have demonstrated their effectiveness in controlling storm and combined sewer overflows. Many regional plans include storage or combinations of storage alternatives as an integral part of the overall control process.

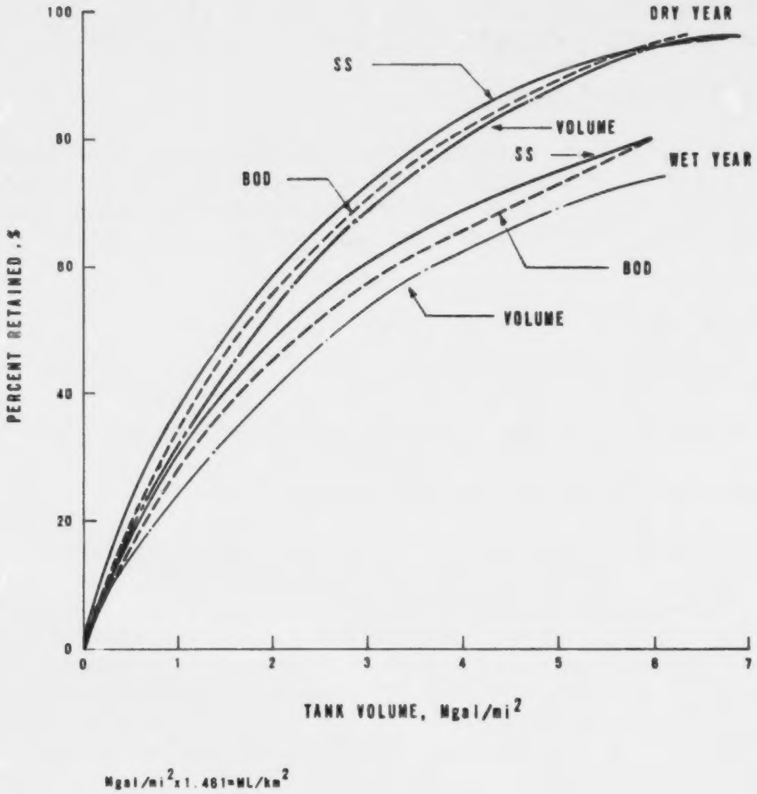


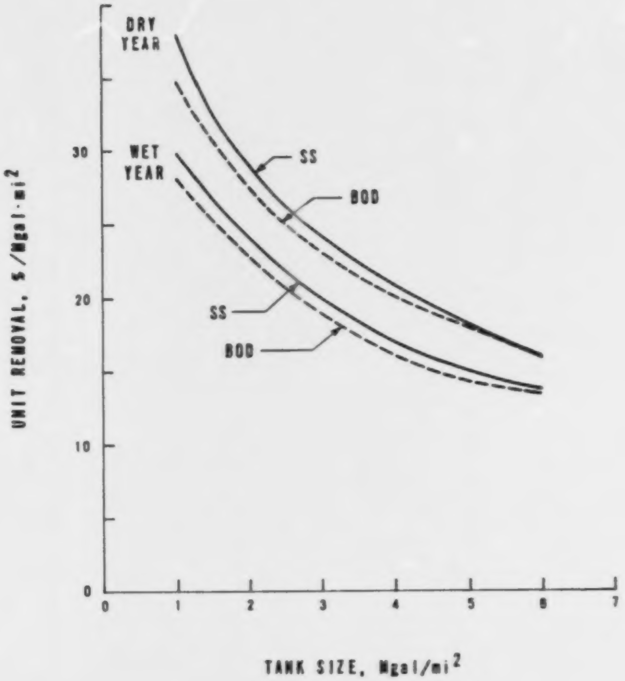
Figure 31. Pollution and volumetric retention versus storage tank volume for wet- and dry-years.

Characteristics and operational performance of the offline storage facilities are described in the literature [2] and are summarized in Table 71.

Offline storage facilities may also be used to relieve dry-weather treatment plants and pumping stations during plant shutdown, by providing auxiliary storage or treatment and preventing flooding of sewers and overflows of raw sewage. The Spring Creek storage facility, during the 2 year period January 1974 to January 1976, received approximately 19 dry-weather flow events as a result of treatment plant shutdowns and malfunctions.

Because of the success of Boston's Cottage Farm Chlorination and Detention Station [2, 17], the Charles River Marginal Conduit Project is currently under design and construction [19, 20]. The purpose of the facility is to avoid pollution between the existing and the proposed dams on the Charles River and to improve water quality for recreational purposes.

The facilities are designed to treat the 1 in 5 year storm flow of 14 150 ML/s (323 Mgal/d), representing an estimated rainfall intensity of 1.19 cm/h (0.47 in./h).



Mgal/mi^2 x 1.461 = ML/km^2

Figure 32. Unit removal efficiencies for combined sewer overflow detention tanks.

The pollution control facilities include:

- Interceptor sewer system, which collects combined sewer overflow to prevent discharge to the Charles River
- Detention and chlorination facilities with effluent pumping station
- Outfall force main to a point below the proposed Charles River Dam

The detention and chlorination facilities will include prescreening, six covered concrete tanks with a total storage capacity of 4.54 ML (1.2 Mgal), and chlorination facilities. System components and operation are similar to Cottage Farm Detention and Chlorination Station.

Characteristic operational parameters for the treatment system are summarized below:

- Frequency of activation approximately 55 times/year or about 4.5 times/month

TABLE 71. DESCRIPTION OF OFFLINE COMBINED SEWER OVERFLOW STORAGE FACILITIES

Location	Description of facilities	System operation	Comments
Akron, Ohio [21]	The underground void space storage tank facilities include a diversion manhole, a 0.38 ML (0.1 Mgal) clarifier, a 3.8 ML (1 Mgal) plastic lined storage basin filled with inert aggregate, an overflow channel to the receiving water, and chlorination facilities. The clarifier chamber contains a tube settler.	Flow is diverted to the clarifier when interceptor capacity is exceeded. As the outlet capacity from the clarifier is exceeded, the flow backs up and overflows to the storage basin. Overflow from the storage basin is discharged to the receiving water. The facilities are dewatered by gravity as interceptor capacity becomes available.	The void space storage concept was used for beneficial land use above the storage basin. Tube settlers do not work effectively in this application. High solids loading to the exposed media of the storage basin may drastically reduce infiltration rates into the bed. Operational since 1974.
Milwaukee, Wisconsin [13] Humboldt Avenue detention facilities	The facilities include a 14.8 ML (3.9 Mgal) covered concrete storage tank, a mechanically cleaned bar screen, a dewatering pumping station, and chlorination facilities. Solids settled in the tank are resuspended by mechanical mixers prior to dewatering.	The tank serves as a settling basin for combined sewer overflow from large storms and totally contains those of smaller storms. Reduction in volume of 67%, suspended solids of 70%, and BOD of 67% were reported for overflow entering the receiving waters.	Effects of a "first flush" condition are evident in this system. Operational since 1969.
Boston, Massachusetts [17] Cottage Farm detention and chlorination station	The detention and chlorination facilities include a 0.38 ML (0.1 Mgal) wet well, and 6 parallel covered concrete storage basins totaling 4.54 ML (1.2 Mgal). The facilities also include prescreening, chlorination, and fine screening of the effluent.	The influent channel is designed to permit sequential filling of the basins as capacity is needed. Reduction in volume of 14%, suspended solids of 45%, and BOD of 42% were reported for overflows entering the Charles River. Small storms that are totally contained and the contents of the basins after large storms are returned to the interceptor for treatment.	The main purpose of this facility is treatment by sedimentation during large overflow events as indicated by the percent volume reduction. Operational since 1971.
New York City, New York [22, 23, 24, 25] Spring Creek	Spring Creek has 6 covered concrete retention basins with a total capacity of 46.9 ML (12.4 Mgal). An additional 49.2 ML (13 Mgal) of storage was estimated to exist in the trunk sewer. The facilities include mechanically cleaned bar racks, chlorination, centrifugal grit separators, and traveling bridge hydraulic sludge washdown devices.	The retention basins provide removal of settleable solids and floatables from overflows, together with disinfection. The system was designed to contain 50% of the summer storms; however, due to the large storage capacity in the trunk sewer, 80 to 85% of the summer storms are contained. Removals from the overflow from storage to the receiving water are: suspended solids, 40%; BOD, 30%, settleable solids, 92%.	Potential overflow is estimated to occur with 0.81 cm (0.32 in.) of rainfall. No estimate of percent volume, suspended solids, and BOD (mass basis) reduction to the receiving water is available. Operational since 1972.

TABLE 71. (Continued)

Location	Description of facilities	System operation	Comments
Chippewa Falls, Wisconsin [18]	The open storage facilities include a diversion structure, bar screen, pumping station, 10.7 ML (2.8 Mgal) open aspahlt-lined storage basin, and a pond overflow structure. Solids removal is facilitated by mechanical street sweepers.	The primary function of the facilities is to store potential overflows, returning them to the interceptor as treatment capacity becomes available. Volume reduction was 93.7%; 95.8% of the suspended solids and 98.2% of the BOD were prevented from entering the Chippewa River.	The dry-weather treatment plants secondary clarifiers were increased in size to retain solids as a result of the increased flow from storage. No detrimental effects on the activated sludge process have been noted as a result of the storage facility. Operational since 1969.
Chicago, Illinois [2, 11, 26]	The deep tunnel project includes tunnels for transmission and storage, mined quarries for additional storage and treatment by sedimentation, and improvements to dry-weather treatment plants to handle the increased wet-weather flow. The tunnel system includes 201 km (125 miles) of deep rock tunnels with an estimated storage capacity of 11 350 ML (2 998 Mgal), and vertical drop shaft entrance structures. Quarry storage is estimated at 156 400 ML (41 315 Mgal). The quarry system includes mechanical aeration, hydraulic dredge sludge removal, and pumping.	The first phase tunnels, storage only without reservoirs, are estimated to reduce overflows from 100 to 10 per year. With only the tunnels in operation, the BOD reduction is expected to be 90% and the volume reduction to the river, 75%. Final plans include instream aeration in addition to the tunnels, reservoirs, and treatment plant improvements to meet dissolved oxygen requirements over the entire 129 km (80 miles) of waterways during critical summer conditions.	Presently three segments of the tunnel system are on line. These include (1) Brookfield La Grange, (2) Crawford Avenue, and (3) Lawrence Avenue. The frequency of overflow using this plan is estimated to be 4 times in 21 years.
Sandusky, Ohio [2, 16]	The offshore underwater storage facility includes a leaping weir regulator, bar screen, diversion weir to allow filling of either or both of the tanks, 2 nylon-reinforced synthetic rubber storage tanks, and dewatering pumps. The storage capacity is 1.36 ML (0.36 Mgal), including the added capacity due to fabric elongation.	Overflow from the combined sewer system is diverted to the underwater storage tanks, and after capacity is reached is directed out the safety overflow. The tanks are dewatered back to the interceptor as treatment capacity becomes available. The system retained 50% of the overflow volume and over 90% of the pollution load. Overflow from the facility occurred approximately 1% of the time.	Operational problems encountered with some of the support equipment included: backflow valves, pressure relief valves, and the tank level control system. Accumulation of silt also created operational problems. Pilot facility operational period was from 1968 to 1969.

TABLE 71. (Concluded)

Location	Description of facilities	System operation	Comments
Washington, D.C. [2, 15]	Combined sewer overflow is stored in 2 underwater nylon reinforced synthetic rubber storage tanks. The storage volume is 0.76 ML (0.2 Mgal). The facilities also include grit chambers, bar screen, comminutor, and return pumping plus instrumentation.	The facilities were designed to contain a portion of the first flush from the combined sewer system. Stored stormwater and solids are pumped to an interceptor after storm flows subside. Approximately 6.06 ML (1.6 Mgal) of overflow has been diverted to storage from 38 storm events.	Operational problems included bag anchoring and tearing, air release devices, and general equipment malfunctions. Compressed air was injected into the storage bags to agitate settled solids for pumping to the interceptor. Operational period January through September 1969.
Columbus, Ohio [3, 12] Whittier Street	Three open concrete storm standby tanks with a total volume of 14.2 ML (3.75 Mgal) were constructed in 1932 with modifications made in 1967. The facilities include traveling bridge sludge scrapers, sludge removal pumps, and regulator structures and gates.	The main processes are pollutant removal by containment and sedimentation. A rainfall of about 0.51 cm (0.2 in.) over the drainage area will put the tank in operation. After storm flows subside, the contained volume and solids are pumped back to the interceptor for further treatment.	Pollutant removal rates due to sedimentation are discussed in the next section. The facilities are operational in present mode since 1967.
Cambridge, Maryland [14]	This underwater storage facility consists of flow metering, bar screen, diversion manholes, a pumping station, and a 0.94 ML (0.248 Mgal) underwater steel storage tank with a flexible rubber diaphragm.	The capacity of the facility is sufficient to totally contain 40 of the 55 storms per year and about 96% of the total annual overflow volume. The facility will capture an estimated 3 240 kg (7 136 lb) of the 3 370 kg (7 435 lb) of BOD in the combined sewer overflow, or about a 96% reduction.	Due to unfavorable public acceptance, the pilot plant facility was removed from the site before completing a satisfactory evaluation. Site selection was proved to be a critical design factor to prevent public disturbance, and includes land use, tidal conditions, and the type of storms to be captured.

- Maximum 1 year return period overflow rate = 5740 L/s (131 Mgal/d)
- Estimated average overflow duration = 7 h
- Estimated maximum overflow duration = 20 h
- Chlorine contact time at design flow = 7.3 min

Projected pollutant removal and operational performance of the Charles River facility are presented in Table 72 [19]. Overall removals expected are: BOD, 61%; suspended solids, 51%; and settleable solids, 61%.

TABLE 72. PROJECTED PERFORMANCE OF CHARLES RIVER MARGINAL CONDUIT STATION [19]

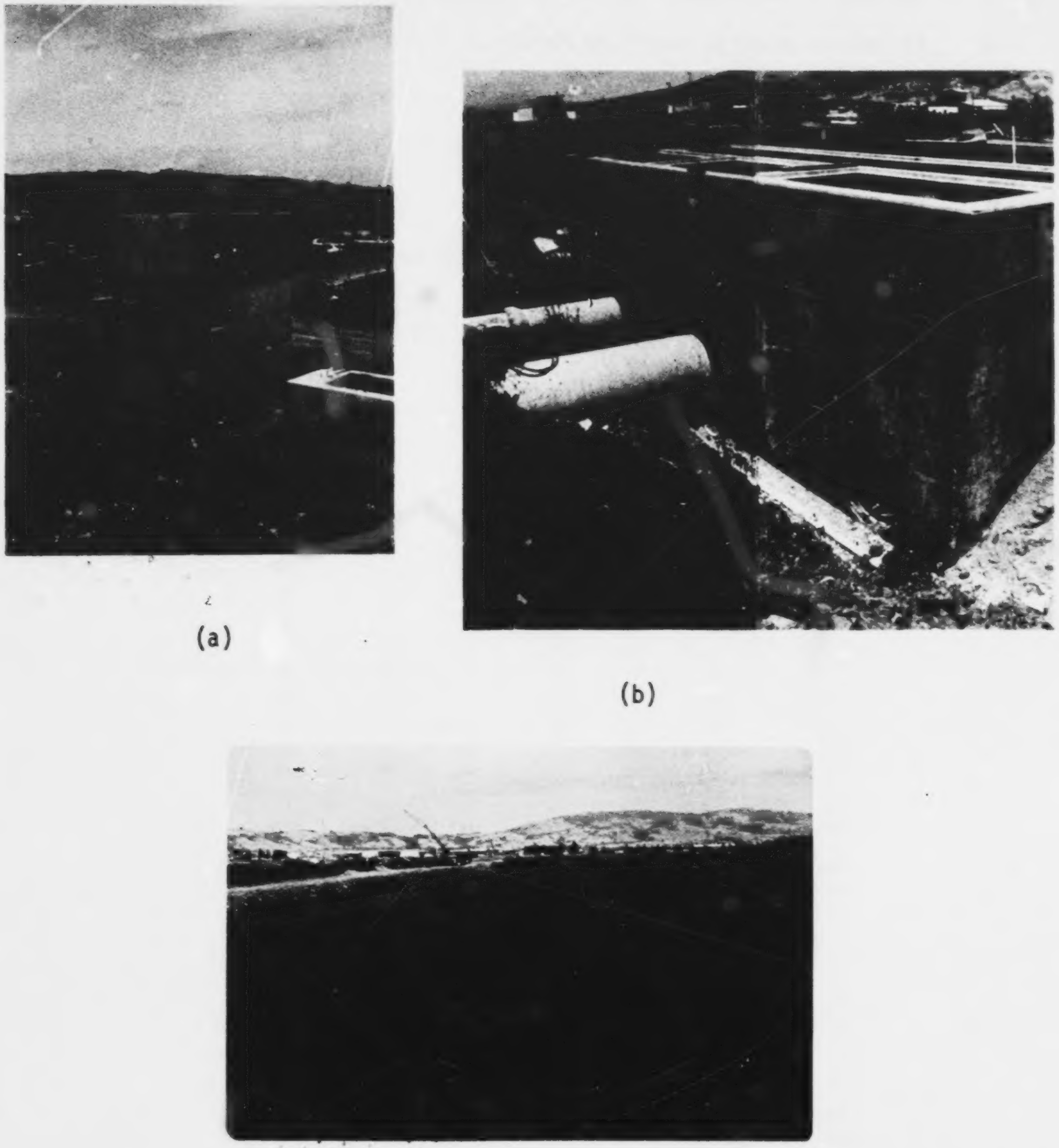
Flows to station			Detention time, min	Reduction, %[a]		
Mgal/d[b]	h/yr	Mgal/yr		BOD	Suspended solids	Settleable solids
1.0	5 260	223	.....	100	100	100
2.0	2 450	206	.....	100	100	100
3.0	550	69	.....	100	100	100
4.0	100	17	.....	100	100	100
5.0	50	11	342.6	88	98	99
15.4	160	103	111.2	40	56	83
17.5	20	15	97.8	36	54	77
25.5	35	37	67.1	28	34	57
37.5	35	55	45.7	24	29	45
50.5	20	42	33.9	20	24	35
65.5	15	41	26.1	17	19	28
81.0	10	34	21.1	15	17	24
98.0	5	21	17.5	14	16	21
131	15	83	13.1	12	15	16
164	5	34	10.4	10	11	14
198	3	25	8.6	8	9	12
230	7	68	7.4	6	7	10
263	3	33	6.5	5	6	8
298	2	25	5.7	3	5	6
323	3	41	5.3	2	4	5
323	12	...	.....	...	...	...
Total	8 760	1 183				

a. Treatment also includes disinfection of all overflows.

b. Flows less than or equal to 4 Mgal/d are assumed totally contained and pumped to Charlestown interceptor to dry-weather treatment. Dry-weather facilities are assumed 100% efficient in these performance estimates.

Mgal/d x 43.808 = L/s
Mgal x 3.785 = ML

An implementation of offline storage was recently constructed in Rohnert Park, California. These facilities will store flows greater than 0.18 m^3/s (4 Mgal/d) from heavily infiltrated sanitary sewers. As capacity at the regional plant becomes available, the storage facilities will be drained. The facilities, under construction, are shown in Figure 33.



(a)

(b)

(c)

Figure 33. Offline storage facilities, Rohnert Park, California
(a) Flow regulator/diversion chamber. (b) Splitter box to storage basins.
(c) Storage basin under construction.

Costs of Offline Storage Facilities--

Updated costs of storage facilities and operation and maintenance costs are presented in Table 73.

Construction cost curves for concrete and earthen storage reservoirs have been developed and are shown in Figure 34 [27]. Earthen reservoir costs include earthwork, liner, paving, seeding, fencing, miscellaneous items, and contingency at 15%. Costs for concrete tanks include concrete and forms, steel, labor, miscellaneous items, and contingency.

PHYSICAL TREATMENT ALTERNATIVES

Physical treatment alternatives are primarily applied for suspended solids removal from wastestreams, and are of particular importance to storm and combined sewer overflow treatment for removal of settleable and suspended solids and flotable material. Physical treatment systems have demonstrated capability to handle high and variable influent concentrations and flowrates and operate independently of other treatment facilities, with the exception of treatment and disposal of the sludge/solids generated from these facilities. The principal disadvantage relates to those periods of time when equipment sits idle during periods of dry weather. When implemented on a dual use basis as either pretreatment or effluent polishing of conventional sanitary sewage treatment plant flows, reduced capital investments may be realized by continuous utilization of the physical treatment system's capacity.

Physical treatment processes that have been demonstrated on either a pilot or prototype scale include: sedimentation and chemical clarification; solids concentration and flow regulation (swirl concentrator/flow regulator) ; screening; dissolved air flotation; high rate filtration; and a relatively new process, magnetic separation [2, 28]. Many prototype level installations employ combinations of the above unit processes to form integrated treatment systems, or use physical treatment processes in conjunction with biological and disinfection to produce desired water quality goals and pollutant removals.

Process descriptions and installations, process performance comparisons, and operational evaluations of the treatment technologies using recent and past data from new and previous demonstration projects form the base for this report on the state-of-the-art update. Design manuals, procedures, and criteria developed in the literature will be used and demonstrated in the illustrative problem sets [2, 29-32].

Chemical treatment operations are included under physical treatment because physical treatment is an integral part of the overall process. Evaluation of chemical additives such as polyelectrolytes, which enhance physical removals are also addressed.

TABLE 73. SUMMARY OF OFFLINE STORAGE COSTS[a]

Location	Storage capacity, Mgal	Drainage area, acres	Capital cost, $	Storage cost, $/gal	Cost per acre, $/acre	Annual operation and maintenance cost, $/yr
Akron, Ohio [21]	1.1	188.5	455 700	0.41	2 420	2 900
Milwaukee, Wisconsin [13]						
Humboldt Avenue	3.9	570	1 774 000	0.45	3 110	51 100
Boston, Massachusetts						
Cottage Farm Detention and Chlorination Station [17][b]	1.3	15 600	6 495 000	5.00	416	80 000
Charles River Marginal Conduit Project [19]	1.2	3 000	9 488 000	7.91	3 160	97 600
New York City, New York [22, 23, 25]						
Spring Creek Auxiliary Water Pollution Control Plant						
Storage	12.39	3 260	11 936 000	0.96	3 660	100 200
Sewer	13.00	.....	....	....	.....	....
	25.39	3 260	11 936 000	0.47	3 660	100 200
Chippewa Falls, Wisconsin [18]						
Storage	2.82	90	744 000	0.26	8 270	2 700
Treatment	....	..	189 000	....	2 100	8 000
	2.82	90	933 000	0.26	10 370	10 700
Chicago, Illinois [2, 11, 26]						
Tunnels and pumping	2,998	240 000	870 000 000	0.29	3 630	....
Reservoirs	41 315	....	682 000 000	0.02	2 840	....
Total storage	44 313	240 000	1 552 000 000	0.04	6 470	....
Treatment	......	....	1 001 000 000	....	4 170	....
	44 313	240 000	2 553 000 000	0.04	10 640	8 700 000
Sandusky, Ohio [16]	0.36	14.86	520 000	1.44	35 000	6 200
Washington, D.C. [2, 15]	0.20	30.0	883 000	4.41	29 430	3 340
Columbus, Ohio [2, 3, 12]						
Whittier Street	3.75	29 250[c]	6 144 000	1.64	210	....
Cambridge, Maryland [14]	0.25	20	320 000	1.28	16 000	14 400

a. ENR 2000.

b. Estimated values; facilities under design and construction.

c. Estimated area.

$/acre x 2.47 = $/ha
$/gal x 0.264 = $/L
Mgal x 3785 = m^3

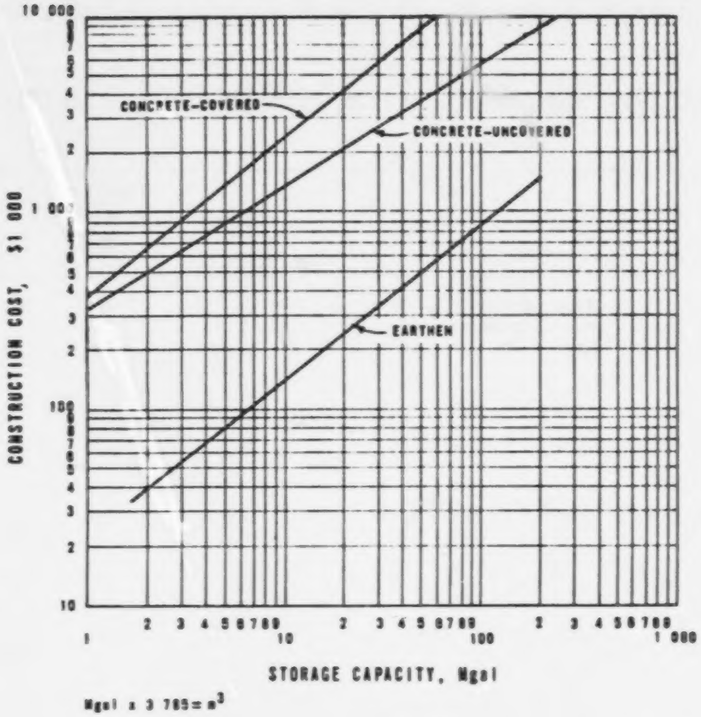


Figure 34. Storage reservoir construction costs (ENR 2000).

Process Description and Facilities Installations

Process descriptions and typical physical treatment installation case histories have been developed [2]. These and several new concepts for treating storm and combined sewer overflows are summarized in the following.

Sedimentation--

Many of the storage installations described in the preceding section also double as primary sedimentation facilities for flows that exceed storage capacity. For several installations, sedimentation is the main treatment process where the total volume captured in storage is less than 50% [12, 13, 17, 19, 22, 33]. Sedimentation has also been used for pretreatment and post-treatment in addition to storage [21, 33]. Most sedimentation/storage facilities also include the following or combinations of the following in the complete treatment system:

- Prescreening
- Postscreening
- Removal of floatables
- Disinfection

Significant sedimentation demonstration and prototype projects are summarized in Table 74.

TABLE 74. SUMMARY OF TYPICAL SEDIMENTATION FACILITIES[a]

Project location	Type of sedimentation facility	Volume of sedimentation tank, Mgal	Maximum design flowrate, Mgal/d	Period of operation
Akron, Ohio [21]	Tube settlers in clarifier and void space storage basin	0.1	149	1974 to present
Boston, Massachusetts				
Cottage Farm Detention and Chlorination Station [17]	Covered concrete tanks	1.3	233	1971 to present
Charles River Marginal Conduit Project [19, 20]	Covered concrete tanks	1.2	323	Under design and construction
Columbus, Ohio [2, 12]				
Whittier Street	Open concrete tanks	3.75	403	1932 to present; modified in 1966
Dallas, Texas [33]				
Bachman Stormwater Plant	Open concrete tanks and tube settlers with waste lime and polymer addition	1.2	28	1971 to present
Milwaukee, Wisconsin				
Humboldt Avenue [13]	Covered concrete tank	3.9	246	1969 to present
New York City, New York				
Spring Creek [2, 22, 25]	Covered concrete tanks	12.4	2900	1972 to present
Saginaw, Michigan [34]				
Weiss Street	Concrete tanks	3.9	...	In design
Hancock Street	Covered concrete tanks	3.5	323	Under construction, 90% complete

a. Treatment of combined sewer overflows except Dallas facility which treats excessive sanitary flows caused by infiltration.

Mgal/ x 3785 = m^3
Mgal/d x 43.808 = L/s

Swirl and Helical Concentrator/Regulators--

Solids concentrator/regulators achieve both quantity and quality control of wastestreams laden with suspended material. The two principal types of control devices developed include the swirl and the helical bend concentrator/regulators [2, 29, 30]. The principal mechanism for dynamic solid/liquid separation is secondary fluid motion attained through long path geometric flow patterns [29].

Helical bend concentrator/regulators have been modeled and design criteria and comparative cost evaluations have been developed [30]. Although no demonstration projects have been implemented in the United States, helical bends appear more practical as inline devices rather than as satellite or offline devices. Swirl concentrators have been modeled and, in several cases, demonstrated for various processes including treatment and flow regulation, grit removal, primary treatment, and erosion control.

- Swirl concentrator/flow regulator--In this application the swirl is used to replace conventional regulators while simultaneously treating combined wastewater by swirl action. During dry weather, sanitary flows are diverted through a channel in the chamber floor into a bottom orifice and discharged to the intercepting sewer. Pumping of the dry-weather flow may be required by limiting hydraulic gradients [35].

- Swirl degritter--The swirl principle has been applied to grit removal for pretreatment prior to other treatment processes and as a degritter for the underflow from a swirl concentrator/regulator [36-38]. Swirl degritters usually have a conical shaped hopper below the circular swirl chamber where the solids accumulate before being discharged.

- Swirl primary separator--The swirl primary separator unit was developed to remove a greater fraction of the suspended solids than the swirl concentrator/regulator does. The configuration of the swirl chamber developed was a conical shaped device with a depth approximately equal to the diameter [36, 39]. The relatively high overflow rates (approximately twice that of conventional sedimentation) used in the swirl design at various levels of suspended solids removal may result in less costly construction and require less space than conventional sedimentation basins.

- Erosion control--A modification of the swirl using a conventional cattle watering tank is being investigated for a portable erosion control device [40]. Erosion and construction site stormwater runoff could be rapidly treated before discharge to the receiving water or retention ponds.

Swirl concentrator and helical bend model studies and demonstration projects are summarized in Table 75 and typical swirl installations are shown in Figure 35.

Screening Alternatives--

Screens have been used to achieve various levels of suspended solids removal contingent with three modes of screening process applications.

- Main treatment - screening is used as the primary treatment process

- Pretreatment - screening is used to remove suspended and coarse solids prior to further treatment to enhance the treatment process or to protect downstream equipment

TABLE 75. SUMMARY OF SWIRL/HELICAL SOLIDS CONCENTRATOR-FLOW REGULATOR FACILITIES

Project location	Type of facility	Unit size diameter, ft[a]	Process application	Period in service
Denver, Colorado [38]	Swirl	6	Sanitary and simulated wet-weather swirl regulator concentrate-pilot scale grit removal	1975 - currently out of service
Lancaster, Pennsylvania [41]	Swirl - Unit 1	24	Solids concentration and flow regulation-prototype.	Under construction
	Swirl - Unit 2	8	Degritter for foul flow from Unit 1 - prototype.	
Lasalle, Quebec, Canada [29, 39, 40]	Swirl	3	Solids concentration and flow regulation - hydraulic model studies with synthetic combined sewage	....
Lasalle, Quebec, Canada [30]	Helical bend	..	Solids concentration and flow regulation - hydraulic model studies with synthetic combined sewage	....
Nantwich, England [30]	Helical bend	..	Solids concentration and flow regulation - prototype.	1971 to present
Rochester, New York [36]	Swirl - Unit 1	3	Degritter - pilot.	1975 to 1976
	Swirl - Unit 2	6	Primary treatment - pilot.	
Syracuse, New York [35, 42]	Swirl	12	Solids concentration and flow regulation - prototype.	1974 to present
Toronto, Ontario, Canada [39]	Swirl	12	Primary treatment of combined sewer overflows and municipal wastewater - pilot.	1975 to early 1977

a. Outer chamber diameter.

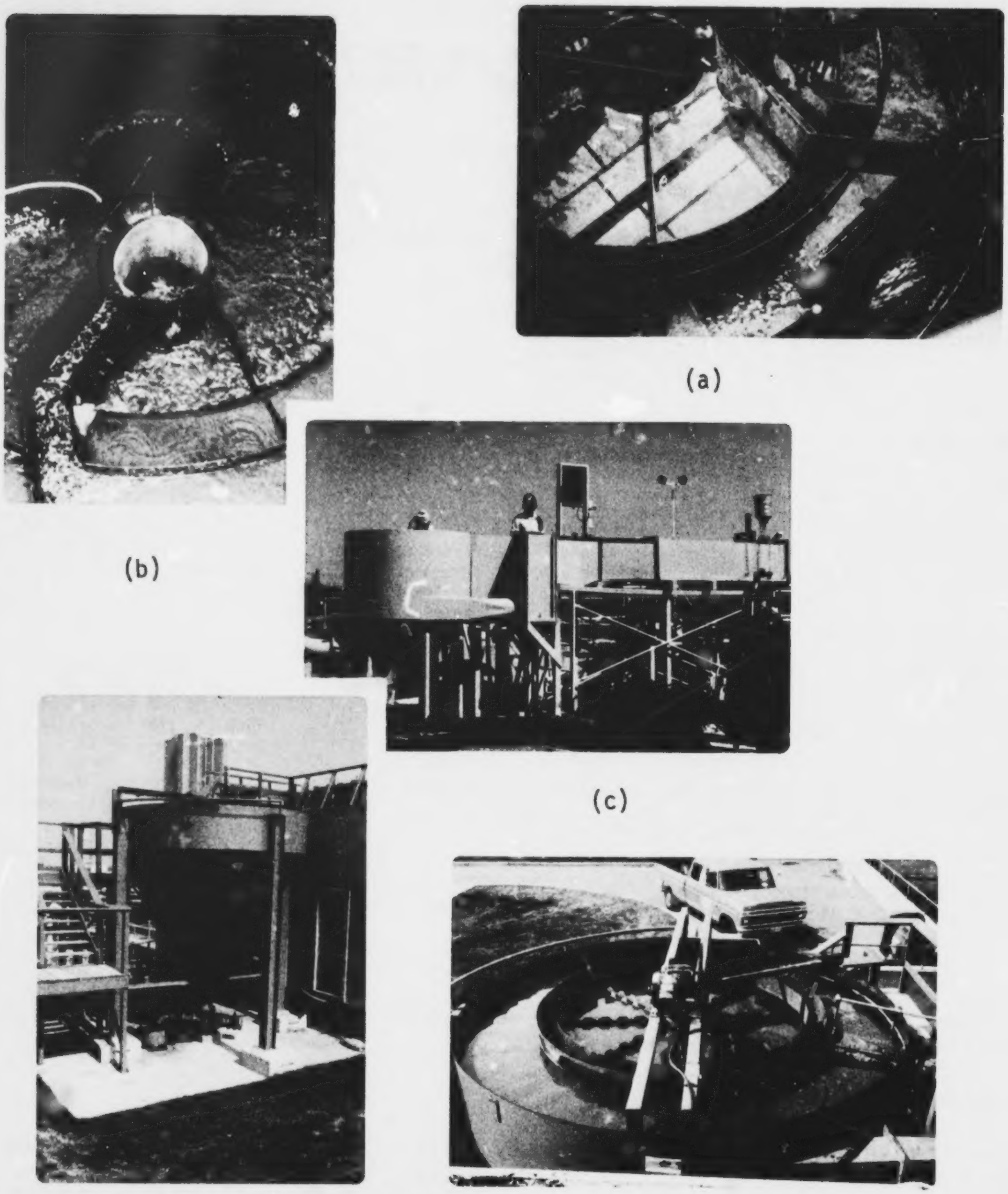
ft x 0.3048 = m

- Dual use - screening provides either main treatment or pretreatment of stormwater and is used as an effluent polisher during periods of dry weather

Several distinct types of screening devices have been developed and used in stormwater treatment and are described in Table 76 [2, 32]. A summary of typical screening installations is presented in Table 77. Photographs of screening installations are shown in Figure 36.

Dissolved Air Flotation--

Dissolved air flotation has been demonstrated as an efficient treatment method to remove suspended solids and floatables such as oil and grease found in combined sewer discharges [43-45]. Solids are removed from the wastestream by small bubbles of air which are released in the reaction tank after depressurization, and rise to the surface carrying the solids. The pressurized flow carrying the dissolved air to the flotation tank is either (1) the entire stormwater flow, (2) a portion of the stormwater flow (split flow pressurization), or (3) recycled dissolved air flotation effluent [2].



(a)

(b)

(c)

(d)

(e)

Figure 35. Swirl concentrator installations.
(a) West Newell Street swirl concentrator/regulator during dry-weather flow-Syracuse, New York. (b) Syracuse swirl concentrator/regulator during a combined sewer overflow. (c) Pilot swirl degritler - Denver, Colorado. (d) Swirl primary separator - Toronto, Canada. (e) Weir arrangement of Toronto swirl.

TABLE 76. DESCRIPTION OF TYPES OF FINE MESH SCREENING DEVICES USED IN COMBINED SEWER OVERFLOW TREATMENT [2, 32]

Type of screen	General description	Process application	Comments
Drum screen	Horizontally mounted cylinder with screen fabric aperture in the range of 100 to 841 microns. Operates at 2 to 7 r/min.	Pretreatment	Solids are trapped on inside of drum and are backwashed to a collection trough.
Microstrainers[a]	Horizontally mounted cylinder with screen fabric aperture in the range of 20 to 70 microns. Operates at 2 to 7 r/min.	Main treatment	Solids are trapped on inside of drum and are backwashed to a collection trough.
Rotostrainer	Horizontally mouned cylinder made of parallel bars perpendicular to axis of drum. Slot spacing in the range of 250 to 2500 microns. Operates at 1 to 10 r/min.	Pretreatment	Solids are retained on surface of drum and are removed by a scraper blade.
Disc strainer	Series of horizontally mounted woven wire discs mounted on a center shaft. Screen aperture in the range of 45 to 500 microns. Operates at 5 to 15 r/min.	Pretreatment, main treatment, or post treatment of concentrated effluents	Unit achieves a 12 to 15% solids cake.
Rotary screen	Vertically aligned drum with screen fabric aperture in the range of 74 to 167 microns. Operates at 30 to 65 r/min.	Main treatment	Splits flow into two distinct streams: unit effluent and concentrate flow, in the proportion of approximately 85:15.
Static screen	Stationary inclined screening surface with slot spacing in the range of 250 to 1600 microns.	Pretreatment	No moving parts. Used for removal of large suspended and settleable solids.

a. A vertically mounted microstrainer is available, which operates totally submerged and operates at approximately 65 r/min. Aperture range 10 to 70 microns. Solids are moved from the screen by a sonic cleaning device.

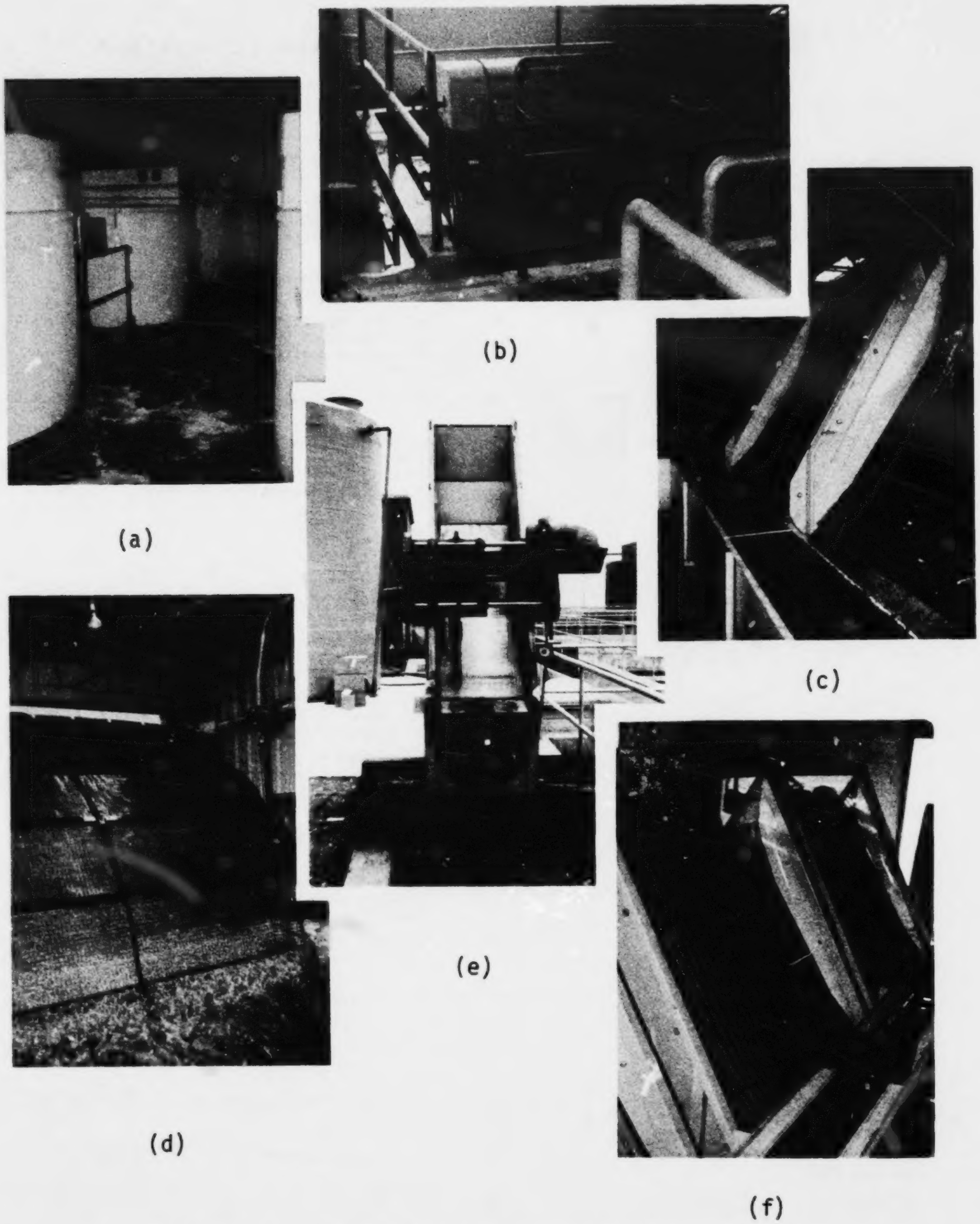
A description of pilot and full-scale demonstration dissolved air flotation facilities is presented in Table 78.

High Rate Filtration--

Several high rate filtration pilot study installations have been demonstrated for control of combined sewer overflow pollution [46, 47]. These facilities have used 15.2 and 76.2 cm (6 and 30 in.) diameter pilot-scale filter columns with anthracite and sand media, together with various dosages of coagulants and polyelectrolytes to develop basic process criteria and optimum operating conditions. Descriptions of the high rate filtration facilities are summarized in Table 79 and shown in Figure 37.

TABLE 77. DESCRIPTION OF TYPICAL SCREENING INSTALLATIONS

Project location	Type of screening equipment	No. of screening units	Screen aperture, microns	Screening application	Period in service
Belleville, Ontario [48]	Rotary screen Static screen Static screen Rotostrainer	1 1 1 1	105 305 762 500	Pilot plant operation to test effectiveness of screening combined sewer overflows	1974 to 1975
Cleveland, Ohio [2, 47]	Drum screen	1	420	Pilot pretreatment to dual media filtration	1970 to 1971
Euclid, Ohio [49]	Microstrainer	4	30	Dual use: dry-weather effluent polishing 98% of time plus main treatment of combined sewer overflow	Under construction
Flint, Michigan [49]	Microstrainer	6	20	Effluent polishing	Under construction
Ft. Wayne, Indiana [50]	Static screen Drum screen Rotary screen	12 1 8	1525 147 105	Parallel screening facility to test effectiveness of various screens, main treatment and pretreatment	1975 to present
Milwaukee, Wisconsin [44, 51]					
Hawley Road					
Test 1	Drum screen	1	297	Pretreatment to dissolved air flotation	1969 to 1972
Test 2	Drum screens Microstrainer	2 1	841 149 63	Sequential screening main treatment, screens operated in series	1971
Test 3	Microstrainer	1	20	Main treatment of combined sewer overflow and dissolved air flotation effluent polishing	1973
Test 4	Drum screen	1	297	Pretreatment to dissolved air flotation with chemical addition	1974
Mt. Clemens, Michigan [52]	Microstrainer	1	20 and 60	Polish pond effluent	1972 to 1975
New York City, New York [23, 53]	Rotostrainer Disc strainer	1 1	297 250 and 420	Pretreatment to high-rate filtration	1975-1976 1976 to present
Norwalk, Connecticut [49]	Microstrainer	6	35 or 70	Dual use: dry-weather effluent polishing and main treatment of combined sewer overflow	Under construction
Oil City, Pennsylvania [49, 54]	Microstrainer	2	35	Dual use: effluent polishing and main treatment	1976 to present
Philadelphia, Pennsylvania [55, 56, 57]	Microstrainer	1	23 and 35	Main treatment with disinfection	1969 to 1974
Racine, Wisconsin [43]					
Site I Site II	Drum screen Drum screen	2 4	297 297	Pretreatment to dissolved air flotation	1973 to present
Site IIA	Drum screen	1	297	Main treatment	
Rochester, New York [36]	Microstrainer	1	70	Pilot main treatment	1975 to 1976
Syracuse, New York [35]	Rotary screen Microstrainer	1 2	105 20 and 71	Pilot main treatment	1974 to present



(b)

(a)

(c)

(e)

(d)

(f)

Figure 36. Stormwater screening installations.
(a) Rotary screens - Ft. Wayne, Indiana. (b) Microscreen, Oil City, Pennsylvania. (c) Static screens - Ft. Wayne, Indiana. (d) Drum screen - Ft. Wayne, Indiana. (e) Static screen with brush cleaner - Belleville, Canada. (f) Static screens - Franklin, Pennsylvania.

TABLE 78. SUMMARY OF TYPICAL DISSOLVED AIR FLOTATION INSTALLATIONS

Project location	No. of tanks	Pressurization mode	Design flow, Mgal/d	Process description	Period of operation
Milwaukee, Wisconsin [44]					
Hawley Road	1	Effluent recycle and split flow	5.0	Pilot main treatment dissolved air flotation system with pretreatment screening and chemical addition.	1969 to 1974
Racine, Wisconsin [43]					
Site I	3	Split flow	14.1	Full scale main treatment utilizing screening for pretreatment.	1973 to present
Site II	8	Split flow	44.4	Full scale main treatment utilizing screening for pretreatment.	1973 to present
San Francisco, California [45]					
Baker Street	2	Either split flow or effluent recycle	24.0	Full scale main treatment with chemical addition; facility has both float and bottom scrapers, with no pretreatment	1970 to present

Mgal/d x 43.808 = L/s

Other Physical and Physical/Chemical Systems--

Bench scale and pilot plant testing of high gradient magnetic separation was evaluated on combined sewer overflows and raw sewage in Boston, Massachusetts [28]. The process involves seeding the wastestream with magnetic iron oxide (magnetite) and adding coagulants and polyelectrolytes to form a floc amenable to removal in a magnetic gradient. The flow is passed through a matrix where the magnetic gradient is induced and the removal occurs. Backwash facilities are included to flush the accumulated floc and particles from the matrix during the backwash cycle when the magnetic gradient is reduced to zero. Removal was found more efficient than sedimentation because the magnetic forces on fine particles may be many times greater than gravitational forces.

A high rate demonstration physical/chemical treatment system for removal of suspended solids, phosphorus, and nitrogen has been evaluated [58]. The process involves inline alum addition and coagulation, polymer addition and flocculation, filtration, and clinophtilolite ion exchange. Suspended solids and phosphorus are removed by alum addition, coagulation, and high rate filtration; and ammonia nitrogen is removed by exchange/adsorption. The system is considered a single unit process, removing phosphorus and ammonia simultaneously.

TABLE 79. DESCRIPTION OF COMBINED SEWER OVERFLOW HIGH RATE FILTRATION PILOT PLANT DEMONSTRATION FACILITIES[a,b]

Project location	Process description	No. of filter columns	Diameter of columns, in.	Pretreatment facilities	Filter media	Period of operation
Cleveland, Ohio [47, 59, 60, 61]	Pilot deep bed, dual media high rate filtration, with chemical addition. Facilities included pretreatment, storage, and filtration.	3 1	6 12	420 micron drum screen	5 ft of No. 3 anthracite over 3 ft of No. 612 sand	1970 to 1971
New York City, New York, Newtown Creek [23, 46, 53]	Pilot deep bed, dual media high rate filtration, with polyelectrolyte addition. Facilities include pretreatment, storage, and filtration. Dry-weather and combined sewer flow is pumped from grit chamber of Newtown Creek plant.	1 2	30 6	420 micron rotostrainer later replaced with a 420 micron disc strainer	5 ft of No. 3 anthracite over 2 ft of No. 612 sand	1975 to present
Rochester, New York [36]	Pilot deep bed, dual media high rate filtration with chemical addition.	3	6	Screening.	5 ft of No.1-1/2 or No.2 anthracite over 3 ft of No.1220 sand	1975 to 1976

a. Systems operated at flux rates ranging from 8 to 30 gal/ft^2 min.

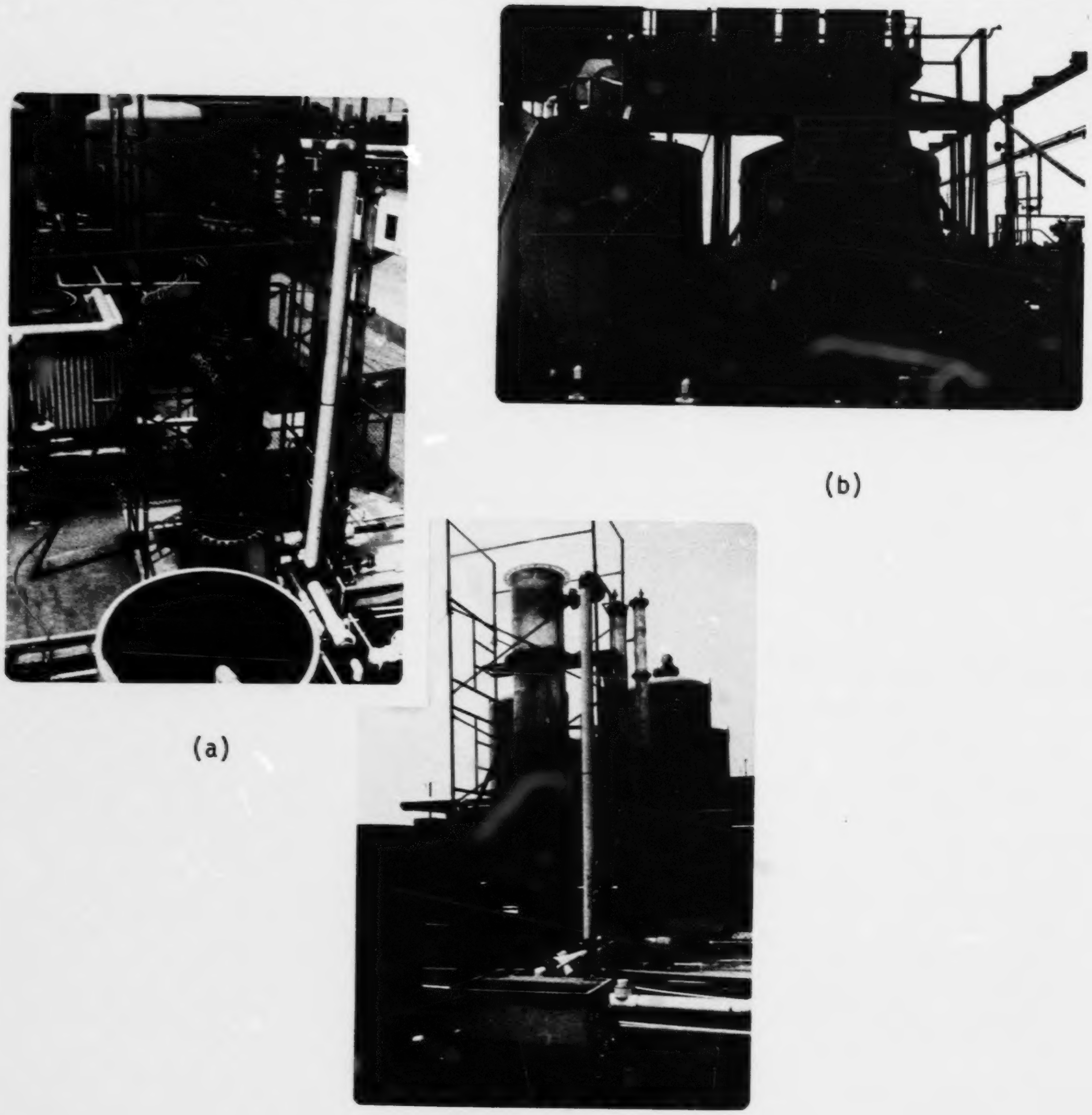
b. High-rate deep-bed filtration has recently, (October-November 1976), been piloted directly on stormwater runoff in the Minnehaha Creek Watershed near Wayzata, Minnesota, under USEPA demonstration grant S-802535. Pretreatment storage was provided to lengthen filtration runs. Publication of results is expected shortly.

in. x 2.54 = cm
ft x 0.305 = m
gal/ft^2·min x 0.679 = L/m^2·s

Other physical/chemical systems employing chemical addition, activated carbon, and filtration are reported in the literature [2], however, most applications involve conventional sewage and no recent information is available for storm and combined sewer overflow applications.

Evaluation of Physical Treatment Technologies

Process performance, including suspended solids and pollutant removal efficiencies; operational problems, both process control and equipment; design criteria; and costs of unit processes are presented as a guide for planners and designers faced with implementing and evaluating complex stormwater treatment systems.



(a)

(b)

(c)

Figure 37. Newtown Creek high rate filtration facilities. (a) 30 in. diameter dual media filter column. (b) Storage tanks prior to screening. (c) View showing 30 in. and two 6 in. filter columns with high rate chlorine contact tank in foreground.

Process Performance--

Pollutant removal was evaluated for the physical treatment processes, and is summarized in Table 80. Removal of suspended solids is used as the key indicator of process performance. Removals of BOD, COD, settleable solids, nitrogen, and phosphorus are reported when available; however, removal efficiencies of these constituents are often erratic and unpredictable, and vary to greater extremes when compared to suspended solids. Ranges of removals are given for those processes where changes in loading rates or other process variables affect removal efficiencies, and sufficient data are available for analyses.

TABLE 80. COMPARISON OF TYPICAL PHYSICAL TREATMENT REMOVAL EFFICIENCIES FOR SELECTED POLLUTANT PARAMETERS

	Percent reduction					
Physical unit process	Suspended solids	BOD_5	COD	Settleable solids	Total phosphorus	Total Kjeldahl nitrogen
Sedimentation						
Without chemicals	20-60	30	34	30-90	20	38
Chemically assisted	68	68	45	.....	..	..
Swirl concentrator/flow regulator	40-60	25-60	..	50-90	..	..
Screening						
Microscreens	50-95	10-50	35	.....	20	30
Drum screen	30-55	10-40	25	60	10	17
Rotary screens	20-35	1-30	15	70-95	12	10
Disc strainers	10-45	5-20	15	.....	..	..
Static screens	5-25	0-20	13	10-60	10	8
Dissolved air flotation[a]	45-85	30-80	55	93[b]	55	35
High rate filtration[c]	50-80	20-55	40	55-95	50	21
High gradient magnetic separation[d]	92-98	90-98	75	99	..	..

a. Process efficiencies include both prescreening and dissolved air flotation with chemical addition.

b. From pilot plant analysis [45]

c. Includes chemical addition.

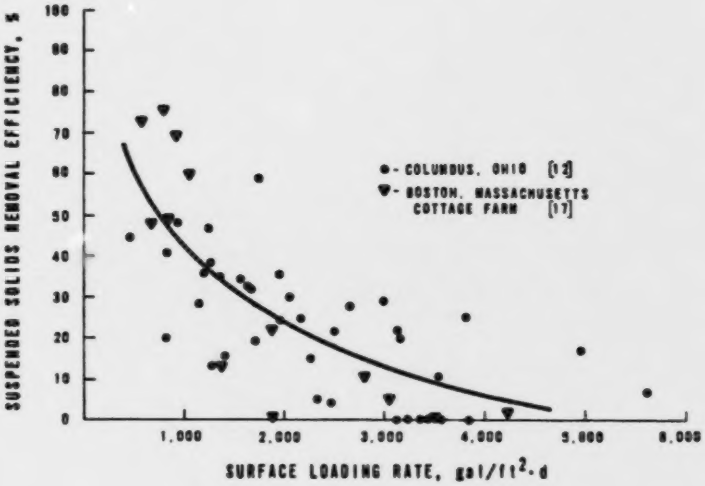
d. From bench scale and small scale pilot plant operation, 1 to 4 L/min (0.26 to 1.06 gal/min).

Process performance curves and removals of other pollutant parameters such as heavy metals have been developed and reported for each unit process. Where possible, these curves reflect changes in removal efficiencies as a result of changing loading rates or critical process variables.

The effects of chemical addition to enhance the physical removal efficiencies have been demonstrated for most unit processes, and generally show increased pollutant removals at higher loading rates. Chemical addition to dissolved air flotation and high rate filtration processes have shown the greatest performance improvement, generally ranging from 20% and higher [43, 44, 47]. Coagulant addition to form a floc is used in high gradient magnetic separation [28].

Typical chemical additives include cationic, anionic, and nonionic polyelectrolytes; and coagulants, such as alum and ferric chloride. Bench and pilot scale studies to select the polymer, coagulant type, and dose rates should be developed for each wastewater and unit process under investigation to optimize pollutant removal rates, as was shown for the high rate filtration project in Cleveland [47].

Sedimentation--Removal of pollutants by sedimentation has shown erratic results for both suspended solids and BOD for stormwater applications. Suspended solids removal as a function of hydraulic loading rates is presented in Figure 38 for typical combined sewer overflow sedimentation facilities. The results represent average suspended solids removals for a storm event, using average hydraulic loading rates during the overflow period. The data scatter is indicative of high and changing hydraulic loading rates and variable influent concentrations.



gal/ft²·d x 40.74 = L/m²·d

Figure 38. Typical suspended solids removal efficiencies for storage/sedimentation facilities without chemical addition.

Typical removal of suspended solids by conventional sanitary sewage settling tanks with surface loading rates at or near 1.698 $m^3/m^2 \cdot h$ (1000 $gal/ft^2 \cdot d$) is approximately 50 to 60% [31]. Similar removals are obtained for stormwater loadings in this range; however, loading rates can vary up to 6 times this value with removals in the range of 0 to 35%.

When removals attributed to total flow capture during small overflow events and that retained by storage/sedimentation during large events are included, removals can range 60% and higher.

Removal of BOD is more erratic than for suspended solids and ranges from 0 to 50% for most loading rates and influent concentrations. Based on typical performance of several sedimentation facilities, average BOD removal rates in excess of 20% are common [12, 17, 33].

Removal of heavy metals, nitrogen, phosphorus, and other constituents by sedimentation has been reported and is summarized in Table 81 [24].

TABLE 81. POLLUTANT REMOVAL FOR VARIOUS CONSTITUENTS BY SEDIMENTATION [24]

Pollutant	Average removal, %
Heavy metals[a]	
Copper	24.1
Chromium	32.3
Nickel	26.6
Zinc	27.2
Lead	30.6
Iron	16.6
Cadmium	38.8
Calcium	19.2
Magnesium	23.5
Sodium	18.5
Potassium	23.5
Mercury	8.4
Nitrogen[b]	
Ammonia	22.1
Organic	50.5
Total Kjeldahl	38.4
Nitrate	15.4
Nitrite	0
Phosphorus[b]	
Total	22.2
Ortho	6.7
Other constituents[b]	
COD	34.4
TOC	21.3
Oil and grease[c]	11.9

a. Average of 10 samples.
b. Average of 2 to 3 samples.
c. Average of 6 samples.

Swirl and Helical Concentrator/Regulators--Suspended solids removals for swirl concentrators average approximately 50% (total mass basis) for combined sewer overflows. In addition to the removal obtained by the physical splitting of flows, as with conventional regulators, the additional 20 to 30% reduction in the suspended solids concentration is attributed to the action of the swirl. Limited tests indicate a BOD mass removal of approximately 67% with a reduction of BOD concentration in the effluent of approximately 47%. However, these tests were conducted at flowrates substantially less than the swirl's design capacity of 0.3 m^3/s (6.8 Mgal/d), and these values may be unrealistically high [35]. Performance of the swirl concentrator/flow regulator is presented in Figure 39, for both overall suspended solids mass removal and concentration reduction. Hydraulic loading rates to the swirl ranged from 8.5 to 51 $m^3/m^2 \cdot h$ (5000 to 30 000 $gal/ft^2 \cdot d$).

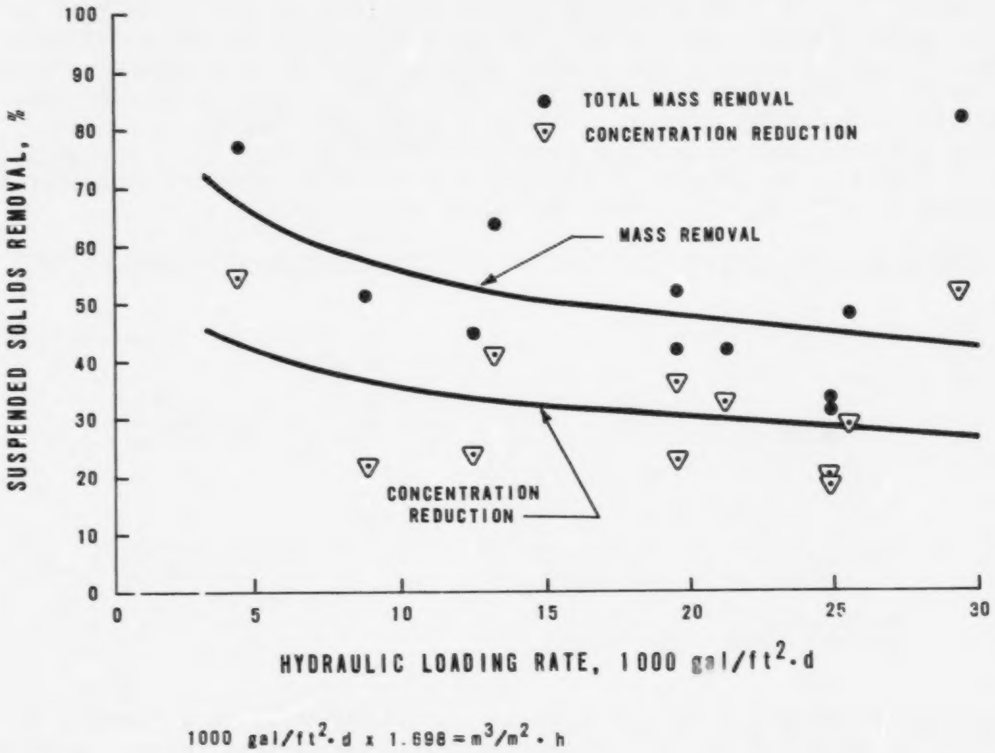


Figure 39. Swirl concentrator/flow regulator suspended solids removal efficiency as a function of hydraulic loading rate [35].

Although no prototype helical bend facilities have been constructed in the United States, it was found through model studies that the helical bend is capable of higher removal efficiencies, with less headloss than the swirl concentrator [30]. The studies also developed design criteria and guidelines for field installations.

Swirl Degritter--Removal of grit was demonstrated on a pilot scale using influent sanitary sewage and sanitary sewage spiked with sand to simulate wet-weather flow conditions [38]. Grit removal efficiencies for flows at less than design capacities ranged from 50 to 87% with an average of approximately 70%. Swirl efficiencies at flows greater than design capacity fall off markedly with an average removal of approximately 34%. Suspended solids removal based on three runs averaged approximatley 7%. The efficiency of removing grit particles of 2.65 S.G. and sizes greater than 0.2 mm was equal to that of conventional sanitary sewage grit removal devices; however, the detention time of the swirl degritter is less than 1 minute as compared to about 3 minutes for conventional aerated grit chambers.

Swirl Primary Separator--A swirl device was also evaluated as a primary separator using sanitary sewage and combined sewer flows at the Humber Wastewater Treatment Plant Toronto, Ontario [39]. The pilot unit was tested at a design flow of 1137 m^3/d (0.3 Mgal/d) and at 1700 m^3/d (0.45 Mgal/d). Approximately 40% suspended solids removal was achieved by the swirl at a hydraulic loading rate of 108 $m^3/m^2 \cdot d$ (2650 $gal/ft^2 \cdot d$) at a detention time of 0.34 hours. In comparison, the conventional settling basins at the Humber facilities had similar suspended solids removal efficiencies at hydraulic loading rates of approximately 81.5 $m^3/m^2 \cdot d$ (2000 $gal/ft^2 \cdot d$) at a detention time of 1.06 h. The treatment efficiency of the swirl primary separator is presented in Table 82 for several pollutant parameters.

TABLE 82. TREATMENT EFFICIENCIES OF A SWIRL PRIMARY SEPARATOR [39][a]

Swirl flow, Mgal/d	Hydraulic loading rate, $gal/ft^2 \cdot d$	Percent removal			
		Suspended solids	Settleable solids	Volatile suspended solids	Fixed suspended solids
0.30	2 650	43	60	46	26
0.45	3 980	25	48	26	22

a. 3.66 m (12 ft) diameter chamber.

Mgal/d x 43.813 = L/s
$gal/ft^2 \cdot d \times 1.698 \times 10^{-3} = m^3/m^2 \cdot h$

Screening--A comparison of suspended solids treatability as a function of influent suspended solids for microstrainers, drum screens, rotary screens, and static screens is presented in Figures 40 through 43. From this comparison, microstrainers show the best performance as a main treatment device; however, hydraulic loading rates for this type of screen are the lowest. For all screens, removal performance tends to improve as influent suspended solids concentrations increase.

In a particle size analysis from the backwash on a 23-micron microstrainer, 2.4% of the particles (by weight) captured were larger than 41 microns and only 3% of the particles were larger than 27 microns. Most of the particles, 81%, were in the range of 7 to 0.07 microns indicating that the effective

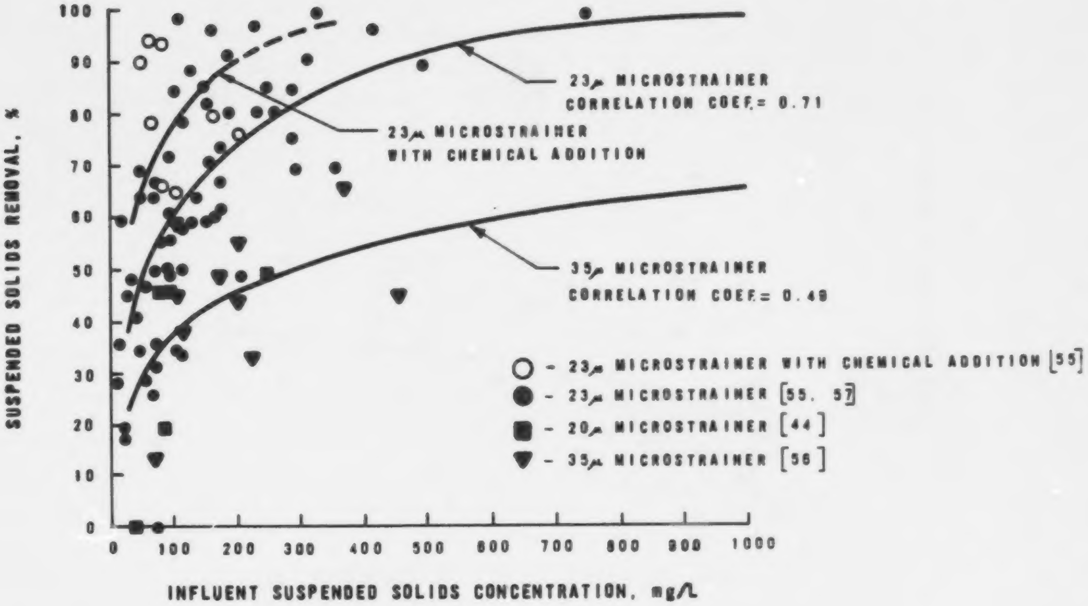


Figure 40. Microstrainer performance as a function of influent suspended solids concentration.

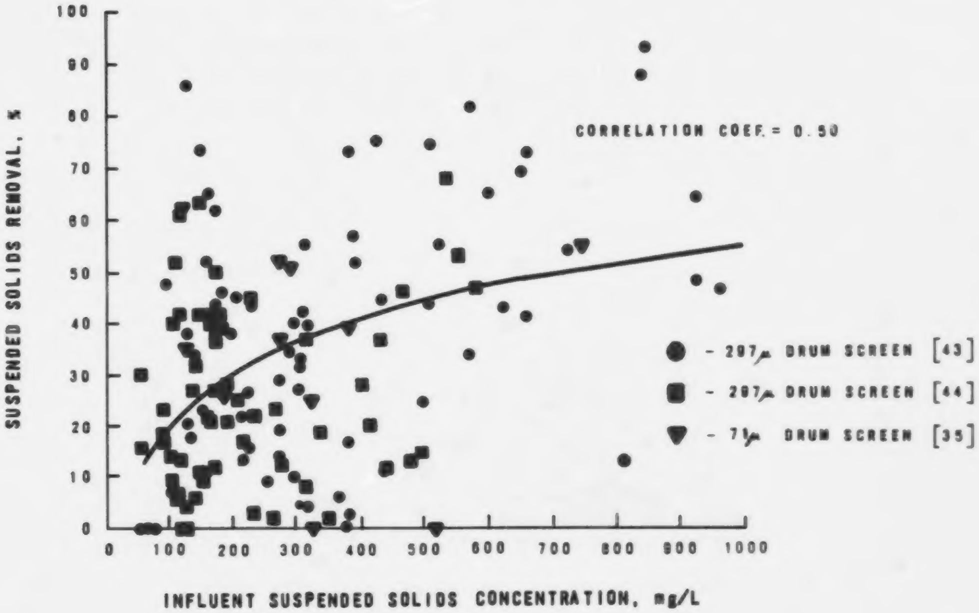


Figure 41. Drum screen performance as a function of influent suspended solids concentration.

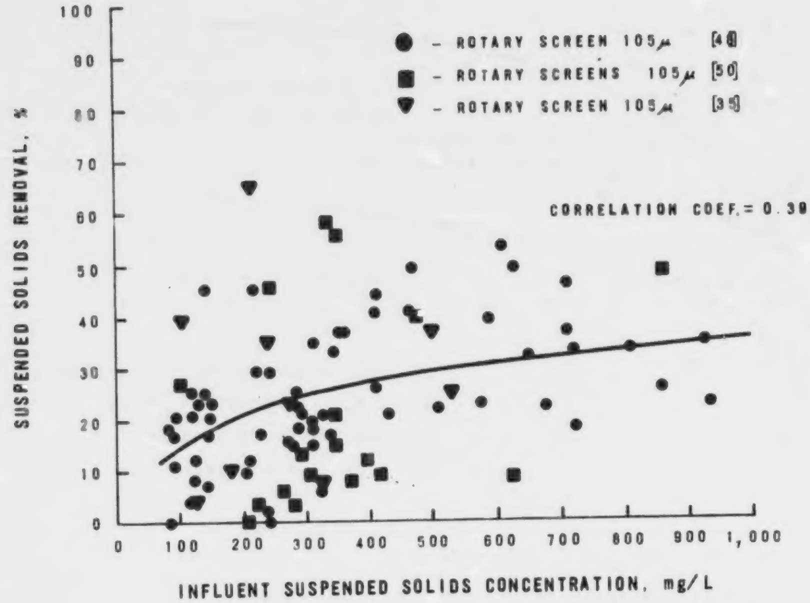


Figure 42. Static screen performance as a function of suspended solids concentration.

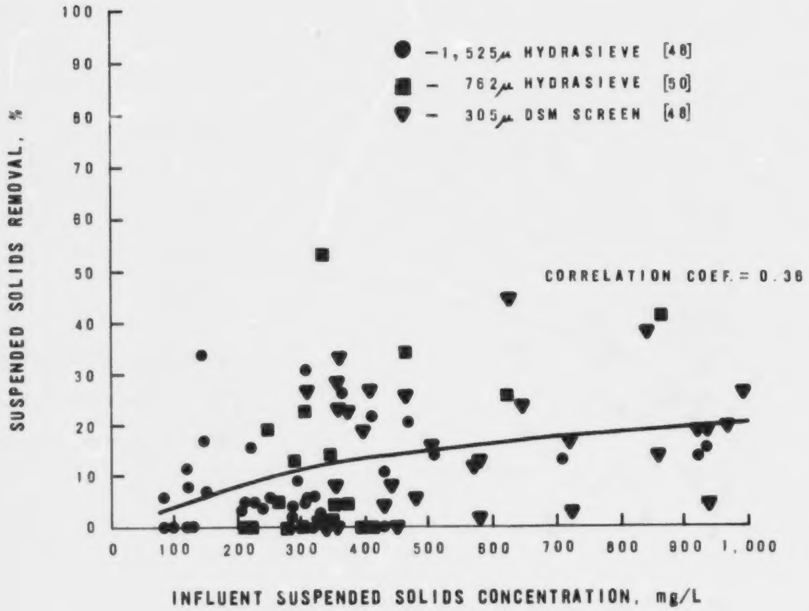


Figure 43. Rotary screen performance as a function of suspended solids concentration.

filtration of a 23 micron screen is better than 23 microns [55]. Microstrainers and drum screens actually develop a mat of screened particles that acts as a strainer retaining particles smaller than the screen aperture. Drum screens, rotary screens, and static screens capture less suspended solids than microstrainers; however, they have been used as pretreatment devices screening out coarse and settleable solids and protecting downstream equipment.

BOD and other pollutant removals are more erratic and have greater data scatters than suspended solids removals. BOD removals for all screens average between 10 and 30%.

Polymer addition to microstrainers improved suspended solids removal efficiency by approximately 10% with increases in average flux rates of 39 to 88 $m^3/m^2 \cdot h$ (16 to 36 $gal/ft^2 \cdot min$). Moderately charged, high molecular weight cationic polyelectrolytes (Betz 1150 and Atlasep 105C) resulting in concentrations between 0.25 to 1.5 mg/L were most suitable for increasing efficiency of the screening operation. The use of polymers also showed increased reduction of volatile suspended solids, COD, and TOC [55].

Dissolved Air Flotation--Dissolved air flotation (DAF) performance has been found to vary with the following control and operational variables [44, 45].

- Surface loading rate to the flotation tank
- Chemical addition
- Influent suspended solids concentration to the flotation tank
- Mode of flow pressurization
- Saturation tank pressure
- Air to solids ratio
- Float skimmer height and speed

A comparison of dissolved air flotation performance efficiency with and without the use of chemicals is presented in Figure 44, incorporating both hydraulic loading rate and influent suspended solids variables as mass solids loading rate. Individual performance data were grouped for each unit solids loading rate and averaged for runs with polymer and/or coagulant addition, and for runs without chemical addition. Limited data were available at high mass loading rates; therefore, individual DAF run data were used instead of average grouped data to represent process efficiency. Data on flotation performance without chemical addition are limited as most applications of this process use chemicals to greatly enhance pollutant removals.

Treatment efficiency on a mass basis showed an increase over the arithmetic mean which gives equal weight to each event without regard to volume treated. Treatment efficiency is usually greater for longer duration high total volume storms than for short duration low volume storms. A comparison of pollutant removals on an arithmetic mean and mass basis is shown in Table 83. The cause of this difference was attributed to the startup lag time of 30 to 45 minutes before good quality effluent was achieved [43]. Higher mass loadings and suspended solids concentrations will also affect DAF efficiency, providing a greater chance for physical contact with the float bubbles.

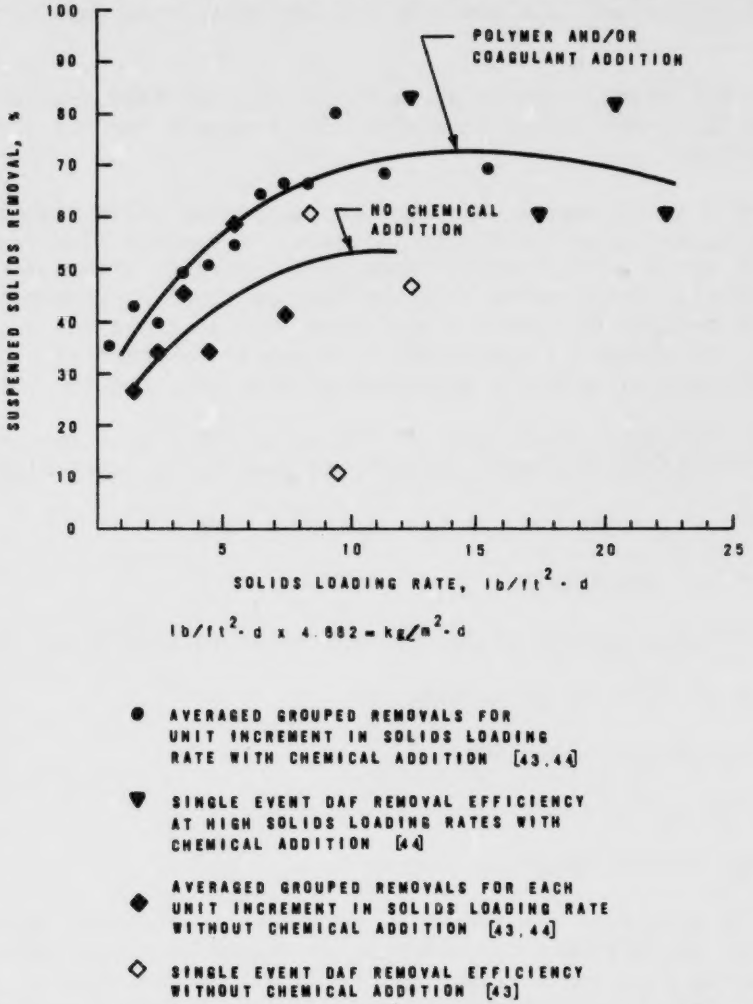


Figure 44. Dissolved air flotation performance as a function of suspended solids loading rate with and without chemical addition.

Low and high hydraulic loading rates affect removal efficiency, as shown in Table 84. Rates were increased from approximately 6.72 to 9.17 $m^3/m^2 \cdot h$ (2.75 to 3.75 $gal/ft^2 \cdot min$). Split flow pressurization of the influent wastewater will result in less hydraulic load to DAF facilities than operating with effluent recycle, which is added to the total wastewater flow entering the tank. Chemical addition to the dissolved air flotation process also affects pollutant removal and should be considered an integral part of the process contributing to higher efficiencies, as shown in Table 85 [44].

TABLE 83. COMPARISON OF POLLUTANT REMOVALS ON AN ARITHMETIC MEAN AND MASS BASIS FOR DISSOLVED AIR FLOTATION FACILITIES AT RACINE, WISCONSIN [43]

Site	Parameter	Avg percent removed Arithmetic mean	Avg percent removed Mass basis
I	BOD	50.1	62.4
	Total organic carbon	47.1	60.0
	Total solids	25.7	28.1
	Suspended solids	59.7	67.6
	Volatile suspended solids	64.7	73.6
	Total phosphorus	46.6	53.2
II	BOD	60.4	69.5
	Total organic carbon	50.4	66.6
	Total solids	37.6	47.2
	Suspended solids	66.1	69.8
	Volatile suspended solids	57.0	67.3
	Total phosphorus	60.3	62.4

TABLE 84. COMPARISON OF DISSOLVED AIR FLOTATION PERFORMANCE FOR LOW AND HIGH HYDRAULIC LOADING RATES [44]

Parameter	Percent removal Low rate, 2.75 gal/ft^2·min	Percent removal High rate, 3.75 gal/ft^2·min
BOD	59	52
COD	57	54
Suspended solids	70	61
Volatile suspended solids	71	64

gal/ft^2·min x 2.445 = m^3/m^2·h

Chemical coagulants, such as alum, ferric chloride, and polymers, are typically used in dissolved air flotation [43-45]. It was found that a ferric chloride dose in the range of 21 to 50 mg/L produced the most significant removals of suspended solids, as summarized in Table 86.

Tests were also conducted on a pilot plant scale evaluating the use of alum as a chemical conditioner. Results of this study showed that alum used singly was more effective than polymer used singly [45]. Optimization of process variables including alum dosage at 75 mg/L, and a hydraulic loading rate at 6.05 m^3/m^2·h (2.49 gal/ft^2·in.), resulted in the pollutant removals summarized in Table 87. Ranges of optimized hydraulic loading rate and alum dosage for various pollutant constituents on an individual basis are presented in Table 88.

TABLE 85. COMPARISON OF DISSOLVED AIR FLOTATION PERFORMANCE WITH AND WITHOUT CHEMICAL ADDITION[a] [44]

	Percent removal	
Parameter	Without chemical flocculant addition	With chemical flocculant addition
BOD	35	60
COD	41	57
Suspended solids	43	71
Volatile suspended solids	48	71
Nitrogen	29	24

a. Includes prescreening.

TABLE 86. OPTIMIZATION OF FERRIC CHLORIDE DOSE FOR DISSOLVED AIR FLOTATION [43]

	Ferric chloride dose, mg/L					
	0	1-10	11-20	21-50	51-70	>70
Mean percent removal	47.2	71.0	70.6	82.2	71.0	71.5
No. of runs considered	5	3	7	5	6	4

High Rate Filtration--Suspended solids removal by high rate dual media filtration was found to vary directly with influent suspended solids concentration and inversely with hydraulic loading rate [2, 47, 59]. Both variables were combined to evaluate process performance of high rate filtration as a function of solids loading rate, as shown in Figure 45. The data represent groupings of hydraulic loading rates from 19.6 to 78 $m^3/m^2 \cdot h$ (8 to 32 $gal/ft^2 \cdot min$). For each grouping of hydraulic loading rates, average influent suspended solids were determined and used to compute average solids loading rate. It was found that there was no correlation between BOD removal and hydraulic loading rate because of the independent variation between dissolved and suspended BOD [47].

Addition of chemicals greatly enhance removal of suspended solids, BOD, phosphorus, and COD [47, 60]. Chemicals include polyelectrolytes, generally resulting in concentrations of approximately 1 mg/L; and coagulants, usually alum, resulting in concentrations of approximately 10 to 30 mg/L. At the

TABLE 87. DISSOLVED AIR FLOTATION PERFORMANCE AT OPTIMIZED PROCESS VARIABLES [45]

Parameter	Concentration		Percent removal
	Influent	Effluent	
Total suspended solids, mg/L	99.5	48.6	51
Settleable solids, mL/L	1.8	0.1	94
Floatable solids, mg/L	1.6	0.5	68
Turbidity, JTU	53.2	17.9	66
BOD, mg/L	32.1	5.9	82
COD, mg/L	97.3	58.4	40
Oil and grease, mg/L	1.8	2.8	0
Kjeldahl nitrogen, mg/L	5.9	3.1	47

Cleveland project, anionic polyelectrolytes proved more effective; however, it is stressed that chemical selection tests be run for each specific site under consideration to obtain optimum removal efficiency. Polyelectrolyte addition increases removals of suspended solids, BOD, and COD by approximately 20 to 35%. A comparison of suspended solids removals with and without polyelectrolyte addition is shown in Figure 46 for specific optimized test runs using the average of 4 to 20 grab samples per run. Addition of phosphorus reducing coagulants such as alum increased removals to approximately 60 to 70% as compared to 40 to 45% without alum addition [47].

Limited tests were also run to determine the reduction of heavy metals by high rate filtration. Results of the tests are presented in Table 89. Removals represent composite samples.

High Gradient Magnetic Separation--High gradient magnetic separation is a new treatment technology applied to storm and combined sewer overflow management. To date only bench scale tests and a pilot plant scale system of 1 to 4 L/m (0.26 to 1.06 gal/m) have been operated, therefore caution must be exercised when scaling up to full scale installations until more information and data are available on treatability and costs for large and variable flowrates and pollutant concentrations.

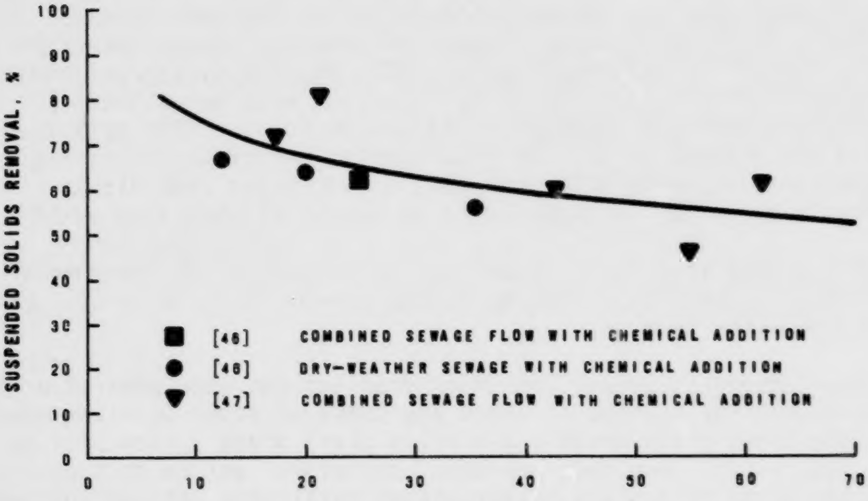
Operational parameters which have the most effect on removal efficiency are coagulant (alum) concentration and pH. Influent suspended solids loading and the magnetic seed concentration affect matrix loading which controls backwash cycling and solids breakthrough. Magnetic field strength above 0.5 kilogauss was not critical to separation efficiency. Ranges of chemical addition for the pilot plant operation include alum at approximately 50 to 120 mg/L, magnetite (size classification 5 to 40 micron) at approximately 0.05 to 0.8 mg/L, and polyelectrolyte at 1 to 3 mg/L.

TABLE 88. RANGE OF HYDRAULIC LOADING RATES AND ALUM DOSAGE FOR SEVERAL POLLUTANT CONSTITUENTS [45]

Stormwater constituent	Alum dosage, mg/L[a]	Hydraulic loading rate, gal/ft^2·min
Total suspended solids	75-150	2.49
Turbidity, JTU	75	2.49
Floatables	75-100	2.49-3.10
Settleable solids, mL/L	50	2.49
BOD	150	1.76-2.49
COD	75-150	2.49
Oil and grease	<100-150	2.78-3.47
Organic nitrogen	Indeterminable	2.49
Ammonium	0-75	2.78

a. Unless otherwise noted.

gal/ft^2·min x 2.445 = m^3/m^2·h



lb/ft^2.d x 4.882 kg/m^2.d

Figure 45. Mean high rate filtration performance as a function of solids loading rate.

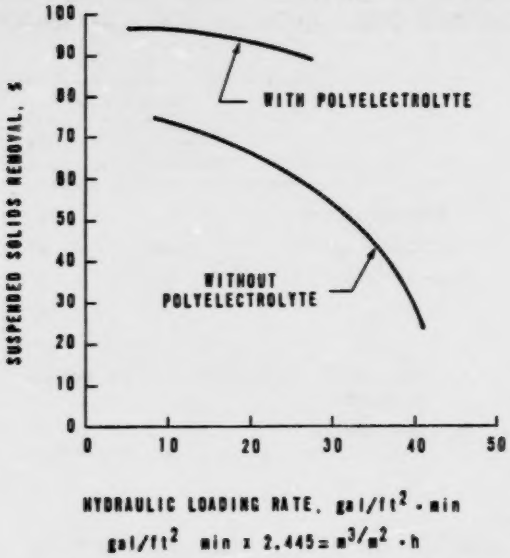


Figure 46. Optimized high rate filtration suspended solids removal with and without polyelectrolyte addition as a function of hydraulic loading rate [47].

TABLE 89. REMOVAL OF HEAVY METALS BY HIGH RATE FILTRATION [46]

	Heavy metal constituent						
	Cadmium	Chromium	Copper	Mercury	Nickel	Lead	Zinc
Average removal, %[a]	56	50	39	0	13	65	48

a. Concentration basis.

Removals of pollutants from bench and pilot scale testing show high removals on a single pass through basis. Pollutant removals of solids, biological material, and heavy metals are summarized in Tables 90 through 92, representing the average of all bench and pilot plant tests [28].

Physical/Chemical Nutrient Removal--The physical/chemical system utilizing inline chemical addition, flocculation, filtration with 1.52 to 2.13 m (5 to 7 ft) of No. 3 Anthrafilt, and ammonia ion exchange through a 1.52 m (5 ft) deep clinoptilolite resin bed showed an 80 to 99% reduction in suspended solids with alum addition at 110 mg/L and polymer addition at 1 mg/L. A 73 micron microstrainer is used as a pretreatment device to remove coarse solids. With aluminum/phosphorus molar ratios larger than 1.0, 80 to 90% phosphorus removal was obtained. Influent ammonia nitrogen concentrations ranging between 0.20 to 0.97 mg/L were reduced to less than 0.20 mg/L [58].

TABLE 90. REMOVAL OF SOLIDS BY HIGH GRADIENT MAGNETIC SEPARATION FOR COMBINED SEWER OVERFLOW AND RAW SEWAGE SAMPLES[a]

Solids parameter	Removal, %[b] Combined sewer overflow	 Raw sewage
Suspended solids	95	91
Settleable solids	99+	99+
Apparent color, PCU	87	82
Turbidity, FTU	93	88

a. All samples concentration basis except as noted.

b. Operated at 1 to 4 L/min (0.26 to 1.06 gal/min), (3 to 12 min residence times).

TABLE 91. REMOVAL OF BIOLOGICAL AND CHEMICAL CONSTITUENTS BY HIGH GRADIENT MAGNETIC SEPARATION

Pollutant parameter	Avg removal, %
BOD	92
COD	74
Total coliforms on EMB agar at 37°C	99.3
Fecal coliforms on EMB agar at 37°C	99.2
Algae	99.9
Virus, bacteriophage T_7	100
Virus, polio	99-100

TABLE 92. REMOVAL OF HEAVY METALS BY HIGH GRADIENT MAGNETIC SEPARATION

	Heavy metal constituent Cadmium	 Chromium	 Copper	 Mercury	 Nickel	 Lead	 Zinc
Average removal, %	>43	>41	53	>71	0-67	0-67	84

Operational Problems--

Many operational problems encountered in stormwater treatment facilities are also common to conventional dry-weather treatment systems. These problems are generally equipment and process control related and include: instrumentation, pumping, level recording and monitoring, and sampling systems. Most problems can be avoided by effective planning and equipment and material selection. Operation and maintenance problems involving installed monitoring and sampling equipment are often able to be field corrected or replaced with more suitable equipment. Several guides for assessing and evaluating flow monitoring and sampling equipment suitability to storm and combined sewer applications are available [62, 63, 64]. Equipment characteristics and requirements, and desirable features are discussed for a compendium of 70 different types of primary flow measurement devices and over 200 models of commercially and custom designed sampling systems.

The following discussion of major problems experienced in operating demonstration and prototype stormwater treatment projects evaluates process application, control, and equipment reliability for several physical process alternatives.

Sedimentation--Application of tube settlers at the Akron, Ohio, and Dallas, Texas, stormwater treatment projects has shown no benefit in improving suspended solids removal [21, 33]. High flowrates at Akron rendered the tube settlers installed at the void space storage project ineffective and deposited large amounts of solids on the exposed media of the storage cell, greatly reducing inflow infiltration rates. Two parameters which affect tube settler performance are (1) rate of flow, and (2) variability of flowrate. Tube settlers operate most effectively with constant flow generally not exceeding loading rates of 9.8 $m^3/m^2 \cdot h$ (4 $gal/ft^2 \cdot min$) [21].

Evaluation of a chemically assisted primary sedimentation process using waste lime from a water purification plant showed marginal benefits in pollutant reduction efficiency. The waste lime sludge contributed to the suspended solids content of the facility effluent. The major operational problem was identified as inadequate control of the waste lime sludge addition to variable flowrates and influent suspended solids concentrations. Polymer addition was also evaluated but results were inconclusive due to inadequate polymer feed equipment [33].

Potential problems for all types of sedimentation facilities are sludge collection and removal, and tank washdown equipment and procedures. Positive sludge removal and cleanup systems are recommended to prevent solids buildup, odors, and excessive maintenance costs [12].

Swirl and Helical Concentrator/Regulators--Although both the swirl and helical concentrator/regulators have no mechanical parts, pumping is often required for swirl installations because of the head requirement through the unit. Potential operational problem may exist with this and other equipment, including control valves, disinfection, flow metering, and sampling equipment commonly used at swirl installations. Automatic flushing or spray washing is also essential to reduce the need for manual cleaning and maintenance after each storm [35].

Screening--Operational and control problems associated with screening have been experienced at most demonstration facilities and are limited to drum screens, microstrainers, and rotary screens. Static screens, since they have no mechanical parts, need little service except for routine cleaning.

Mechanical problems have been reported with the operation of drum screens and microstrainers. Slippage and reduced speed of rotation of the drum were experienced under increased headloss across the drum and under hydraulic loadings. Main bearing support failures, roller bearing support failures, V-belt drive slippage, screen panel support damage, and excessive vibration were also reported [50, 54].

Typical operational problems include screen blinding due to oil and grease buildup and biological growth on the screen panels. These problems have been reduced by adding cleaning agents and solvents to the backwash cleaning system for oil and grease, and by providing ultraviolet light to control the growth of biological slimes.

The principal operational problems attributed to rotary screens include: screen life; backwash cycling; turbulence and high impact velocities of the water striking the screen panels; breaking up solids; and floc, if chemicals are used, forcing them through the screen.

Screen failure is the result of high rotational speeds, high hydraulic loading rates, and impact and abrasion by coarse solid objects in the influent feed. By varying flowrates and rotational speeds, ultimate screen life was increased from an average of 34.3 hours to 346 hours, with an average of approximately 3.5 repairs per screen [65]. A statistical analysis for the Ft. Wayne, Indiana, facilities revealed that the mean time between failures for any one rotary screen unit was 13.25 hours. The useful life for each screen was 30.5 hours. It is expected that with the addition of coarse screening prior to rotary screening, screen life can be increased to several hundred hours [50].

Backwash cycling in the automatic mode when specified hydraulic splits are reached has caused major hydraulic problems and flooding by backwashing all unit simultaneously. This problem can be solved by putting backwash cycling on a timer and providing lockouts allowing only one unit out of service at a time [50].

Rotary screens create two flow streams, a clarified effluent, and a concentrate flow in the ratio of approximately 85:15. The concentrate flow may require additional facilities for collection disposal of solids.

Other Physical Treatment Alternatives--Dissolved air flotation, high rate filtration, and other physical/chemical treatments systems have operational problems similar to conventional treatment systems. These systems generally use some type of physical pretreatment. Process efficiency depends on chemical addition in proportion to flow, suspended solids or other influent pollutant concentrations, therefore requiring complicated chemical feed and metering equipment.

Operational difficulties for dissolved air flotation which affect process performance include:

- Destruction of air bubble-particle aggregates in the inlet zone of the tank because of increasing hydraulic loading and turbulence

- Hydraulic overloading of the effluent launders

- Breakup of float by excessive agitation of the liquid surface in the flotation tank

- Hydraulic short-circuiting in the flotation tank

The major operational problem for high rate filtration is the accumulation of compressible organic solids on the filter media, greatly reducing hydraulic capacity and reducing the length of filter runs. These problems are overcome by using pretreatment devices such as drum screens or disc strainers, which effectively remove coarse and organic solids [47, 66].

Possible operational problems for high gradient magnetic separation include sludge/solids generation and disposal. Further testing is required to determine sludge and mass balances or the possibility of magnetic seed regeneration. Recycle of the magnetic seed up to 5 to 6 times may be a possibility.

Design Criteria--

The design criteria developed for the physical treatment alternatives [2] can be used to determine and evaluate the size and the resulting costs of the various unit processes, or combinations of unit processes, in planning stormwater treatment systems. The design criteria also represent a range of parameters by which process efficiency may be altered to achieve specific treatment requirements, or to optimize the process in terms of cost effectiveness.

Commonly practiced treatment processes, such as sedimentation, are applied at extreme design limits to handle the variable characteristics of storm and combined sewer overflows. Design criteria for other processes such as the swirl concentrator/regulator have been developed through model studies [29], with some field verification to back up the design rational. Design criteria for process equipment such as screens, dissolved air flotation, and high gradient magnetic separators are recommended by the manufacturers and are supported by field operating data.

Sedimentation--The basic design criteria developed for offline storage facilitites also apply when using the storage facility as a sedimentation basin. The principal design criteria affecting both the physical size and treatment efficiency include (1) hydraulic detention time, and (2) surface loading rate. Because stormwater flowrate and volume vary over time and are different for each storm, sedimentation facilities must be designed to operate over a broad range of loadings, as shown in Table 93 for selected sedimentation installations. It is recommended that sedimentation detention times at peak design flowrates be in the range of approximately 20 to 30 minutes, however, some facilities have been designed as low as 6 minutes. Peak hydraulic loading rates generally average 11.9 $m^3/m^2 \cdot h$ (7000 $gal/ft^2 \cdot d$). Normal loading rates for most storm overflows are in the range of 3.4 to 5.1 $m^3/m^2 \cdot h$ (2000 to 3000 $gal/ft^2 \cdot d$).

TABLE 93. AVERAGE AND EXTREME DESIGN VALUES FOR SELECTED SEDIMENTATION FACILITIES

Project location	Surface loading rate, $gal/ft^2 \cdot d$		Detention time, min	
	Average	Peak design	Average	Minimum at peak flow
Boston, Massachusetts [17]				
Cottage Farm Detention and chlorination facility	2 000	6 000	117	8
Columbus, Ohio [12]				
Whittier Street	2 120	7 100	63	13
Dallas, Texas [33][a]				
Bachman stormwater plant	715	1 728	159	66
Milwaukee, Wisconsin [13]				
Humboldt Avenue	.....	7 800	...	23
New York City, New York [25]				
Spring Creek Auxiliary Pollution Control Facilities	4 000[b]	20 300	20[c]	6
Saginaw, Michigan [34]				
Hancock Street	.....	7 260	...	15

a. Chemically assisted sedimentation with waste lime sludge.

b. Estimate to occur at less than this value 98% of the time.

c. Detention time is 20 min or greater, 98% of the time.

$gal/ft^2 \cdot d \times 1.698 \times 10^{-3} = m^3/m^2 \cdot h$

The large hydraulic loading rate for New York City's sedimentation facility does not account for the large volume of trunk sewer storage which will greatly reduce the peak flow to the facility [25]. The average loading rate for this installation was estimated from a rainfall intensity of 1.27 cm/h (0.5 in./h). Rainfall intensities less than this amount were estimated to occur for over 98% of the time. Using a runoff coefficient of 0.5, surface loading rates less than 6.8 $m^3/m^2 \cdot h$ (4000 $gal/ft^2 \cdot d$) are estimated to occur 98% of the time.

Swirl and Helical Concentrator/Regulators--Design criteria for the swirl and helical concentrator/regulators have been developed through hydraulic model studies using synthesized combined sewage particles [29, 30]. Both units are designed as a function of the inlet diameter. For the swirl concentrator, the inlet diameter is related to the chamber diameter by curves developed for different efficiencies of settleable solids removal [29]. Some problems do exist, however, when using the design curves for inlet dimensions and flows that do not fall within the range presented in the curves. Additional modeling and study are required to expand the curve usability to meet flow and inlet sizes encountered in field applications. It is also recommended that emergency side overflow weirs be provided in the swirl design [41, 67]. A general design layout of the swirl concentator/regulator is shown in Figure 47.

General design layouts of the helical bend concentrator/regulator are shown in Figure 48. Model studies showed that the optimum interior angle was approximately 60 degrees. The design details for the helical bend are for 100% grit (0.2 mm, S.G. = 2.65) removal.

Swirl Degritter--Design criteria and design curves for discharges from 0.1 to 2.5 m^3/s (2.3 to 57 Mgal/d) have been developed through hydraulic model studies using synthetic grit particles [68].

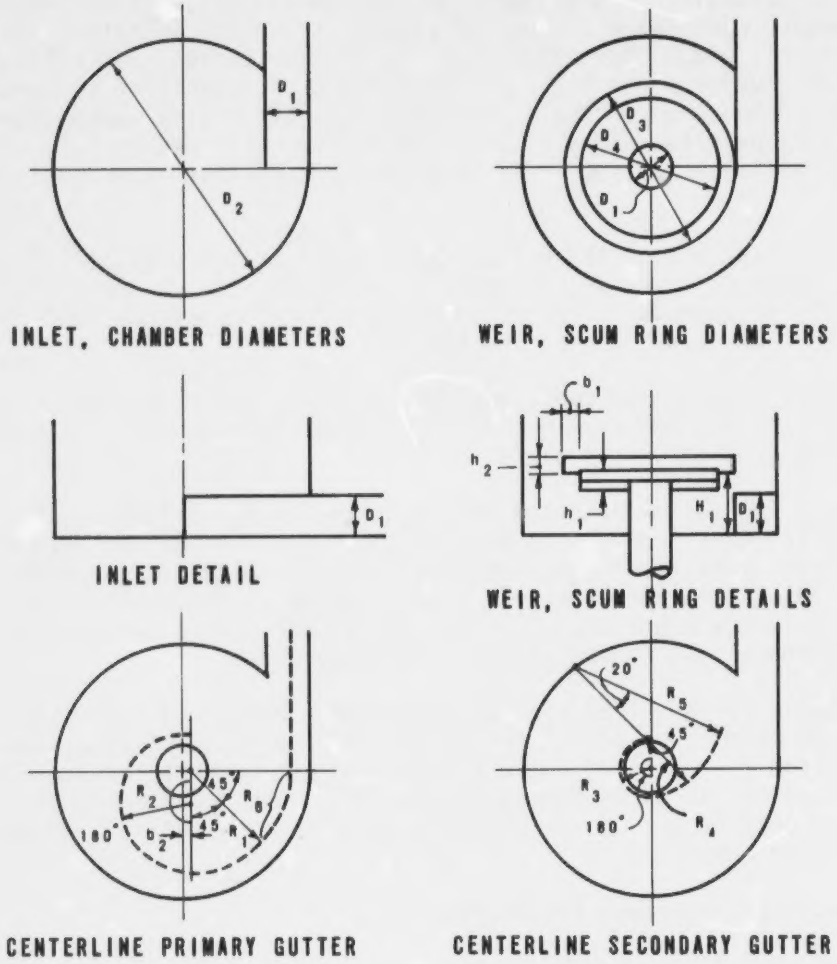
Swirl Primary Separator--Detailed design instructions, criteria, and design curves for flowrates from 0.5 to 500 L/s (0.01 to 11.4 Mgal/d) have been developed from hydraulic and mathematical models for the swirl primary separator [39]. The conical shaped configuration of the device utilizes a height equal to its diameter, which should enhance sludge concentrations but also may decrease cost competitiveness in large sizes.

Design Criteria for Physical Process Equipment--Design and operational criteria have been reported for the screening alternatives, dissolved air flotation, high rate filtration, and high gradient magnetic separators, and are summarized in Tables 94 through 99 [2, 28]. The design parameters generally reflect ranges of operational limits experienced in a number of field installations.

Costs of Physical Treatment Alternatives--

Construction cost and average operation and maintenance costs for physical treatment processes are presented as a guide for planners to determine the relative economic impacts of various treatment alternatives on a first cut basis. Detailed cost studies are still required, including local conditions or changing design requirements, when preparing estimates for specific application or final selection of alternatives.

Construction cost and operation and maintenance cost curves have been developed for combined sewer overflow treatment facilities ranging in size from 0.2 to 8.8 m^3/s (5 to 200 Mgal/d), and for storage facilities ranging in size from 3.8 to 908 ML (1 to 240 Mgal) [27]. Facilities include: storage, sedimentation, screening, swirl concentrator/regulator, dissolved air flotation, filtration, disinfection, chemical feed systems, flow measurement, and raw wastewater and sludge pumping stations. Costs represented by these curves do not include cost of land, engineering, and contingencies.



D_1
D_2
INLET, CHAMBER DIAMETERS
D_3
D_4
D_1
WEIR, SCUM RING DIAMETERS
D_1
INLET DETAIL
b_1
h_2
h_1
H_1
D_1
WEIR, SCUM RING DETAILS
45°
R_2
R_6
180°
b_2
45
R_1
CENTERLINE PRIMARY GUTTER
20°
R_5
45°
R_3
180°
R_4
CENTERLINE SECONDARY GUTTER

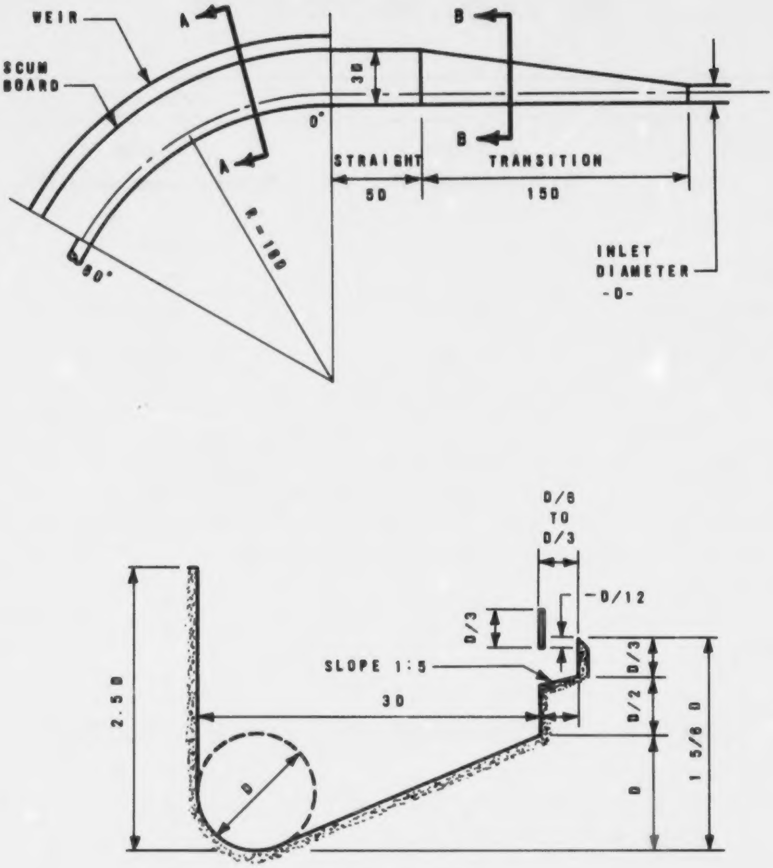
D_1 - USE EFFICIENCY
D_2 - CURVES [016]
H_1 - $D_2/4$
D_3 - $2/3\ D_2$

D_4 - $5/9\ D_2$
h_1 - $D_1/2$
h_2 - $D_1/3$
b_1 - $D_2/16$

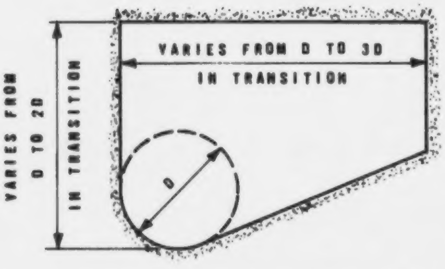
R_1 - $7/16\ D_2$
R_2 - $D_2/4$
R_3 - $5/48\ D_2$
R_4 - $3/16\ D_2$

R_5 - $11/16\ D_2$
R_6 - CURVE SMOOTHED
IN TO MEET
INLET CENTERLINE

Figure 47. General swirl concentrator/regulator design details [29].


WEIR
SCUM BOARD
A
A
B
B
3D
0°
60°
R = 10D
STRAIGHT
5D
TRANSITION
15D
INLET DIAMETER -D-
D/6 TO D/3
D/12
D/3
SLOPE 1:5
3D
2.5D
D
D/3
D/2
D
1 5/6 D

SECTION A-A


VARIES FROM D TO 3D IN TRANSITION
VARIES FROM D TO 2D IN TRANSITION
D

TYPICAL SECTION B-B

Figure 48. Recommended plan and section details for the helical bend concentrator/regulator [30].

TABLE 94. DESIGN PARAMETERS FOR MICROSTRAINERS, DRUM SCREENS, AND DISC SCREENS

Parameter	Microstrainers	Drum screen	Disc screens
Screen aperture, microns	23-100	100-420	45-500
Screen material	Stainless steel or plastic	Stainless steel or plastic	wire cloth
Drum speed, r/min			
Speed range	2-7	2-7	5-15
Recommended speed	5	5	....
Submergence of drum, %	60-80	60-70	50
Flux rate, gal/min per ft^2 of submerged screen	10-45	20-50	20-25
Headloss, in.	10-24	6-24	18-24
Backwash			
Volume, % of inflow	0.5-3	0.5-3	....[a]
Pressure, lb/in^2	30-50	30-50	....

a. Unit's waste product is a solids cake of 12 to 15% solids content.

$gal/min \cdot ft^2 \times 2.445 = m^3/h \cdot m^2$
in. x 2.54 = cm
ft x 0.305 = cm
$lb/in.^2 \times 0.0703 = kg/cm^2$

TABLE 95. DESIGN PARAMETERS FOR ROTARY SCREENS

Screen aperture, microns	
Range	74-167
Recommended aperture	105
Screen material	Stainless steel or plastic
Peripheral speed of screen, ft/s	14-16
Drum speed, r/min	
Range	30-65
Recommended speed	55
Flux rate, $gal/ft^2 \cdot min$	70-150
Hydraulic efficiency, % of inflow	75-90
Backwash	
Volume, % of inflow	0.02-2.5
Pressure, lb/in^2	50

ft/s x 0.305 = m/s
$gal/ft^2 \cdot min \times 2.445 = m^3/m^2 \cdot h$
$lb/in^2 \times 0.0703 = kg/cm^2$

TABLE 96. DESIGN PARAMETERS FOR STATIC SCREENS

Hydraulic loading, gal/min per ft of width	100-180
Incline of screens, degrees from vertical	35[a]
Slot space, microns	250-1 600
Automatic controls	None

a. Bauer Hydrasieves (TM) have 3-stage slopes on each screen: 25°, 35°, and 45°.

gal/min·ft x 0.207 = L/m·s

TABLE 97. DESIGN PARAMETERS FOR DISSOLVED AIR FLOTATION

Overflow rate, gal/ft^2·min	
Low rate	1.3-4.0
High rate	4.0-10.0
Horizontal velocity, ft/min	1.3-3.8
Detention time, min	
Flotation cell range	10-60
Flotation cell average	25
Saturation tank	1-3
Mixing chamber	1
Pressurized flow, % of total flow	
Split flow pressurization	20-30
Effluent recycle pressurization	25-45
Air to pressurized flow ratio, standard ft^3/min·100 gal	1.0
Air to solids ratio	0.05-0.35
Pressure in saturation tank, lb/in^2	40-70
Float	
Volume, % of total flow	0.75-1.4
Solids concentration, % dry weight basis	1-2

gal/ft^2·min x 2.445 = m^3/m^2·h
ft/min x 0.00508 = m/s
standard ft^3/min·100 gal x 0.00747 = m^3/min·100 L
lb/in^2 x 0.0703 = kg/m^2

TABLE 98. DESIGN PARAMETERS FOR DUAL MEDIA HIGH RATE FILTRATION

Filter media depth, ft	
No. 3 anthracite	4-5
No. 612 sand	2-3
Effective size, mm	
Anthracite	4
Sand	2
Flux rate, gal/ft^2·min	
Range	8-40
Design	24
Headloss, ft	5-30
Backwash	
Volume, % of inflow	4
Air	
Rate, standard ft^3/min·ft^2	10
Time, min	10
Water	
Rate, gal/ft^2·min	60
Time, min	15-20

ft x 0.305 = m
gal/ft^2·min x 2.445 = m^3/m^2·h
standard ft^3/min·ft^2 x 0.305 = m^3/m^2·min

TABLE 99. PRELIMINARY DESIGN PARAMETERS FOR HIGH GRADIENT MAGNETIC SEPARATORS [28]

Magnetic field strength, kG[a]	0.5-1.5
Maximum flux rate, gal/ft^2·min	100
Minimum detention time, min	3
Matrix loading, g solids/g of matrix fiber	0.1-0.5
Magnetite addition, mg/L	100-500
Magnetite to suspended solids ratio	0.4-3.0
Alum addition, mg/L	
Range	90-120
Average	100
Polyelectrolyte addition, mg/L	0.5-1.0

a. kG = kilogauss

gal/ft^2·m x 2.445 = m^3/m^2·h

Representative facilities costs are presented in the following paragraphs, utilizing actual construction cost bid tabulations and estimates from stormwater facilitiestogether with data used to develop the detailed cost curves [27]. All costs are adjusted to the ENR 2000 cost index to be compatible with values presented in "Urban Stormwater Management and Technology, An Assessment" [2].

A general comparison of the cost of the various physical treatment processes is presented in Table 100. The ranges of costs were estimated, and in some cases, adjusted to a plant capacity of 1.10 m^3/s (25 Mgal/d). Average capacity costs reflect an approximate cost for a treatment process group indicating relative differences in magnitude between other processes.

TABLE 100. SUMMARY OF AVERAGE CONSTRUCTION COSTS FOR 25 Mgal/d PHYSICAL TREATMENT FACILITIES[a]

Physical treatment process	Construction costs, $	Average cost, $/Mgal·d
Sedimentation[b]	238 000-850 000	23 000
Swirl concentrator/ regulator[c]	50 000-65 000	4 500[d]
Screening[e]	400 000-600 000	19 000
Dissolved air flotation[f]	600 000-1 200 000	34 000
High rate filtration	1 400 000-1 700 000	58 000[g]
High gradient magnetic separation	2 113 000	84 500

a. ENR 2000.

b. Adjusted to 25 Mgal/d costs.

c. Range for 90 and 100% grit removal.

d. Based on a 12 Mgal/d facility.

e. Estimates include supplemented pumping where used.

f. Based on hydraulic loading rate of 5 760 $gal/ft^2 \cdot d$--includes processing and chemical addition facilities.

g. Based on hydraulic loading rate of 24 $gal/ft^2 \cdot min$--includes prescreening and chemical addition facilities.

Mgal/d x 0.0438 = m^3/s
$gal/ft^2 \cdot d \times 1.698 \times 10^{-3} = m^3/m^2 \cdot h$
$gal/ft^2 \cdot min \times 2.445 = m^3/m^2 \cdot h$

Costs of Sedimentation Facilities--Costs of sedimentation facilities are summarized in Table 101, with flow capacities based on a theoretical 30 minute detention time to provide an equal basis of comparison. Actual detention times based on maximum flowrates range from approximately 8 minutes [17] to over 1 hour [33].

Concentrator/Regulators Costs--Costs of swirl concentrator/regulators are based on estimates and actual construction costs excluding land costs, bypass sewers, and engineering and contingencies [27, 30]. Construction costs for swirl facilities are presented in Figure 49 for swirl chamber diameters of 3.05 to 15.2 m (10 to 50 ft).

TABLE 101. SUMMARY OF COSTS OF TYPICAL SEDIMENTATION FACILITIES[a]

Project location	Flow capacity, Mgal/d[b]	Construction costs, $/Mgal·d	Cost, $/acre	Annual operation and maintenance cost, $/Mgal·d
Boston, Massachusetts				
Cottage Farm [17]	62.4	104 000	420	1 280
Charles River [19, 20]	57.6	164 700	3 160	1 690
Columbus, Ohio [12]				
Whittier Street	180.0	34 000	210	.....
Dallas, Texas [33]				
Bachman Stormwater Plant	57.6	31 900	.....	720
Milwaukee, Wisconsin [13]				
Humboldt Avenue	187.0	9 500	3 100	270
New York City, New York				
Spring Creek [2, 22, 25][c]	595.0	20 060	3 660	170
Saginaw, Michigan [34]	168.0	19 760	2 040	200

a. ENR = 2000.

b. Based on 30 minute detention time.

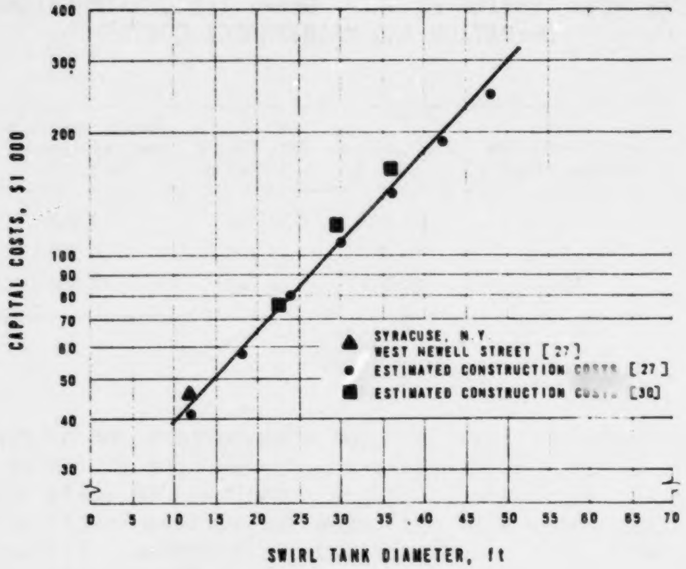
c. Neglecting 13.0 Mgal of trunk sewer storage.

Mgal/d x 0.0438 = m^3/s
Mgal x 3785 = m^3
acre x 0.405 = ha

Operation and maintenance costs have been developed based on the number of overflow events per year, and on an annual manhour basis [27]. Actual operation and maintenance costs have been reported at approximately $2000 per year (ENR 2000) for the West Newell Street installation at Syracuse, New York [69].

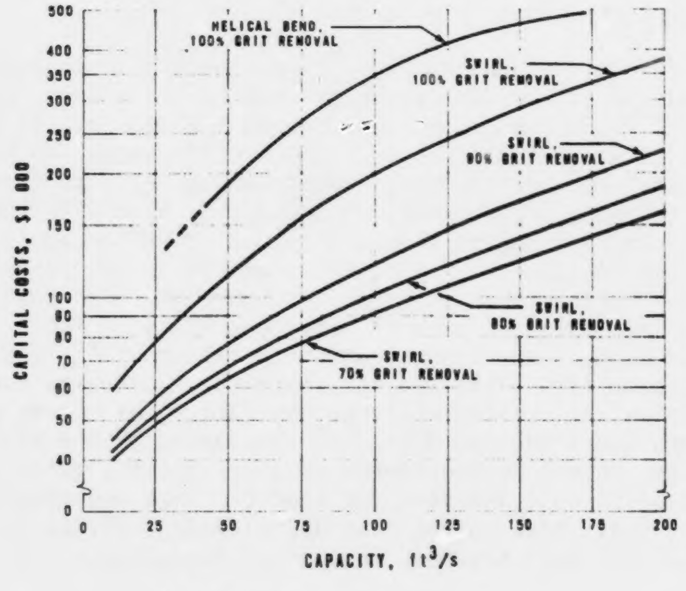
A comparison of costs for various levels of grit removal for the swirl concentrator/regulator and the helical bend concentrator/regulator is presented in Figure 50. Swirl design was based on figures generated from model studies, with ENR 2000 costs applied from Figure 49. Only in cases where low probability peak flows are being considered should designs based on 80 and 70% grit removal be considered for use [29].

Swirl Degritter Costs--Swirl degritter construction and operation and maintenance costs were estimated for units with capacities of 44, 131, and 438 L/s (1, 3, and 10 Mgal/d) and are presented in Table 102 [38]. The estimates include miscellaneous costs for piping, weirs, plates, and costs for a grit washer and screw conveyor. Engineering and contingencies are not included. Operation and maintenance costs include labor, materials and supplies, and energy costs.



ft x 0.305 = m

Figure 49. Estimated construction cost for swirl concentrator/regulators (ENR 2000).



$ft^3/s \times 28.32 = L/s$

Figure 50. Comparison of costs for swirl and helical bend concentrator/regulator for various degrees of grit removal (ENR 2000).

TABLE 102. ESTIMATED SWIRL DEGRITTER CONSTRUCTION AND OPERATION AND MAINTENANCE COSTS[a]

Swirl degritter capacity, Mgal/d	Construction cost, $	Cost/Mgal·d, $/Mgal·d	Annual operation and maintenance cost, $/yr
1	29 100	29 100	3 600
3	33 400	11 100	5 900
10	40 800	4 100	10 600

a. ENR = 2000.

Mgal/d x 0.0438 = m^3/s

Costs of Screening Facilities--Costs of drum screens and microstrainers, rotary screens, and static screens are based on cost estimates from actual demonstration scale facilities, and are summarized in Table 103. For several installations, costs were also estimated for various levels of capacity based on the configuration of the demonstrated installation. Capital construction costs for all screening alternatives range from \$78 to \$166/m^3·h (\$12 300 to \$26 000/Mgal/d) and average approximately \$120/m^3·h (\$19 000/Mgal/d). The range of capital cost values generally reflects special construction methods, type of building, and/or support facilities such as separate pumping stations or structural and architectural requirements at specific sites. Operation and maintenance costs average approximately \$0.013/m^3 (\$0.05/1000 gal), and range from approximately \$0.005 to \$0.026/m^3 (\$0.02 to \$0.10/1000 gal) for static screens and all other types of screens.

Costs of Dissolved Air Flotation Facilities--Costs of dissolved air flotation facilities used for stormwater treatment have varied widely, from approximately \$127 and \$165/m^3·h (\$20 000 and \$26 000/Mgal·d) [43, 44], to over \$443/m^3·h (\$70 000/Mgal·d) [45]. These differences can be attributed to special structural and architectural requirements, requirements for pretreatment, and more importantly, to the design hydraulic loading rate which can change the cost per design flow capacity by a factor up to 3. For this reason, costs for dissolved air flotation facilities are presented as a function of tank surface area as shown in Figure 51. The cost curves represent data developed for several different sizes of facilities based on the experienced cost of the demonstration facilities [45], and cost curves developed from data from dissolved air flotation facilities used in conventional solids thickening applications [27]. The curves present a range of cost with the San Francisco data [45] considered on the high side. These costs, therefore, should be considered as a preliminary guide and should be followed by detailed cost analysis for specific site applications. Operation and maintenance costs have ranged from approximately \$0.013 to \$0.059/m^3 (\$0.05 to %0.22/1000 gal) treated, including pretreatment [43, 44].

Costs of High Rate Filtration--Costs of high rate filtration facilities are summarized in Table 104 [28]. These costs are based on facilities similarly designed to that of the Cleveland demonstration project and include a low lift

TABLE 103. COST SUMMARY OF SELECTED SCREENING ALTERNATIVES[a]

Project location	Type of screen	Screening capacity, Mgal/d	Capital cost, $	Cost, $/Mgal·d	Annual operation and maintenance cost, $/1000 gal
Belleville, Ontario [48][a]	Rotary screen	1.8	33 500	18 600	0.083
		5.4	97 700	17 900	0.083
		7.2	128 400	17 800	0.083
	Static screen	0.75	14 900	19 900	0.042
		5.3	95 600	18 200	0.042
		7.5	130 700	17 400	0.042
Cleveland, Ohio [47][b,c]	Drum screen	25	608 500	24 340	.....
		50	887 800	17 750	.....
		100	1 745 200	17 450	.....
		200	3 340 300	16 700	.....
Ft. Wayne, Indiana [50]	Static screen	18	272 400	15 100	0.020
	Drum screen	18	254 300	14 100	0.039
	Rotary screen	38	584 700	15 400	0.046
Mt. Clemens Michigan [52]	Microstrainer	1.0	26 200	26 200	.....
Philadelphia, Pennsylvania [55]	Microstrainer with chemical addition	7.4	90 800	12 270	0.048
	Microstrainer without chemical addition	7.4	147 900	19 980	0.049
Racine, Wisconsin [43]	Drum screen	3.9	22 600	5 800	.....
Seattle, Washington [70][b]	Rotary screen	25	600 000	24 000	0.098
Syracuse, New York [27][c]	Rotary screen	5	129 500	25 900	.....
	Drum screen	10	257 000	25 700	.....

a. ENR 2000.

b. Estimated costs for several sizes of facilities.

c. Estimates include supplemental pumping stations and appurtenances.

Mgal/d x 0.0438 = m^3/s
$/1000 gal x 0.264 = $\$/m^3$

pumping station, pretreatment by 420 micron drum screens, and chemical addition facilities [47]. Operation and maintenance costs are based on 300 hours of operation per year.

Costs of High Gradient Magnetic Separation--Costs of high gradient magnetic separation have been evaluated for a 1.10 m^3/s (25 Mgal/d) facility and are summarized in Table 105 [28]. Capital costs include pretreatment, chemical addition, thickening and dewatering equipment, pumps, backflush system, instrumentation, and disinfection system. Operation and maintenance costs include chemicals, labor, electrical utilities, and maintenance.

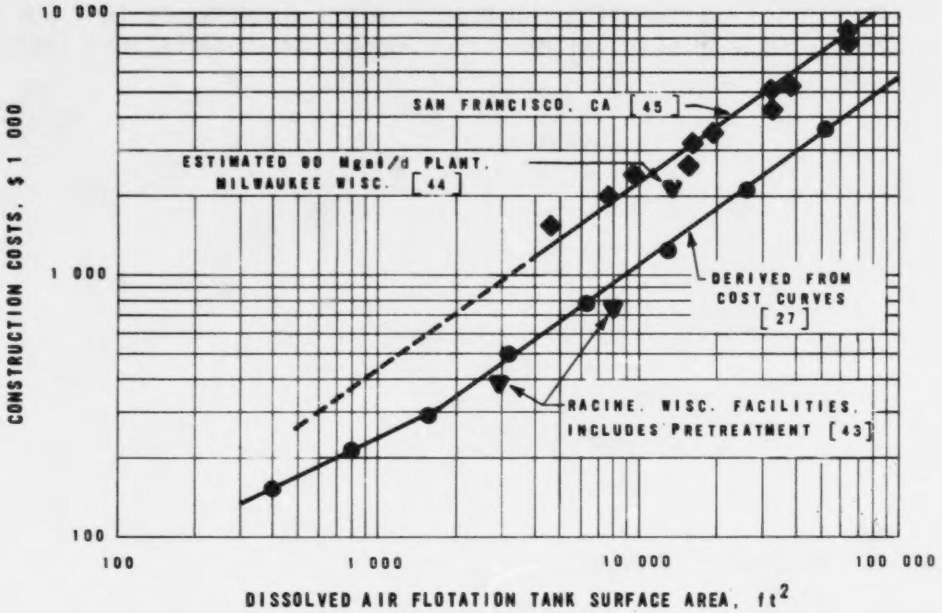
Costs of Physical/Chemical Treatment Systems--Costs of complete physical/chemical treatment systems including chemical clarification and chemical recovery, carbon adsorption, and activated carbon regeneration have been developed [2]. Costs of these facilities for a 1.10 m^3/s (25 Mgal/d) plant range from approximately \$4 000 000 to over \$50 000 000 or \$3 600 000 to over \$45 000 000/$m^3 \cdot s$ (\$160 000 to over \$2 000 000/Mgal·d). Operation and maintenance costs range from approximately \$0.01 to \$0.69/m^3 (\$0.03 to \$0.26/1000 gal) treated. Many of the treatment components include physical treatment processes previously described.

Physical Treatment Systems

The various physical treatment alternatives are generally combined with storage and, in some cases with each other, to form integrated full scale storm and combined sewer management and control systems. In most treatment situations, storage/detention should be considered an essential element of the overall plan to provide flow equalization and/or primary treatment.

Screening devices, particularly microstrainers and drum screens, have been the most widely used physical treatment device in physical treatment systems. They have been used primarily as pretreatment devices to such processes as dissolved air flotation and high rate filtration. By using a 297 micron drum screen before dissolved air flotation, the overall suspended solids removal was increased from an average of 50 to approximately 70% for the facilities in Racine, Wisconsin [43]. Similar results have been obtained by using prescreening with a 420 micron drum screen before high rate filtration [47]. Screens have also been used as effluent polishers after sedimentation and dissolved air flotation.

Typical physical treatment process schematics are shown in Figures 52 through 54. These process systems are the most commonly found for the control of stormwater on a demonstration and full-scale plant level.



NOTE: ft^2 x 0.0929 = m^2
Mgal/d x 0.0438 = m^3/s

Figure 51. Cost of dissolved air flotation facilities.

TABLE 104. SUMMARY OF COSTS FOR DUAL MEDIA HIGH RATE FILTRATION FACILITIES [47]

Plant capacity, Mgal/d	Construction costs, $[b]		Construction costs, $/Mgal·d		Operation and maintenance costs, $	
	24 gal/ft^2·min	16 gal/ft^2·min	24 gal/ft^2·min	16 gal/ft·min	24 gal/ft^2·min	16 gal/ft^2·min
25	1 440 000	1 680 000	57 600	67 200	44 000	45 000
50	2 170 000	2 620 000	43 400	52 400	55 000	57 000
100	3 980 000	4 860 000	39 800	48 600	98 000	102 000
200	6 760 000	8 020 000	33 800	40 100	129 000	134 000

a. ENR 2000

b. Includes low lift pumping station, prescreening, and chemical addition facilities; and excludes engineering and administration.

Mgal/d x 0.0438 = m^3/s
gal/ft^2·min x 2.445 = m^3/m^2·min

TABLE 105. CONSTRUCTION AND OPERATION AND MAINTENANCE COST FOR A 25 Mgal/d HIGH GRADIENT MAGNETIC SEPARATION INSTALLATION[a] [28]

Construction cost	
Total, $	2 113 000
$/Mgal·d	84 500
Operation and maintenance cost	
$/yr	544 000
$/1 000 gal treated	0.12

a. ENR 2000.

Mgal/d x 0.0438 = m^3/s
1 000 gal x 3.785 = m^3

BIOLOGICAL TREATMENT ALTERNATIVES

Biological treatment is a means of removing organic pollutants from wastewater streams, and can be accomplished either aerobically or anaerobically. Several biological processes have been applied to combined sewer overflow treatment, including: contact stabilization, trickling filters, rotating biological contactors (RBC), and treatment lagoons [2].

Biological systems must be operated continuously to maintain an active biomass or be able to borrow the biomass from a system which does operate continuously. This and the high initial capital costs are serious drawbacks in utilizing biological systems in stormwater treatment.

Development and testing of new biological treatment processes and further demonstration of established stormwater biological systems at other locations have not been attempted beyond the original demonstration facilities. Complete descriptions, including design criteria, process performance costs, and facilities descriptions, have previously been evaluated [2]. The following contains a summary of each process, using updated information and data, when available, of completed biological facilities.

Process Description and Installations

Descriptions of the biological processes used to control the organic pollutants found in stormwater are summarized in Table 106. These biological systems are generally located adjacent to conventional biological facilities for a source of biomass, with the possible exception of treatment lagoons. Contact stabilization, trickling filters, and RBCs require supplemental treatment, usually final clarification, to remove the biological solids generated by the process. Effluent from treatment lagoons may also require additional treatment for control of algae or floatable solids. Descriptions of typical biological treatment installations are summarized in Table 107.

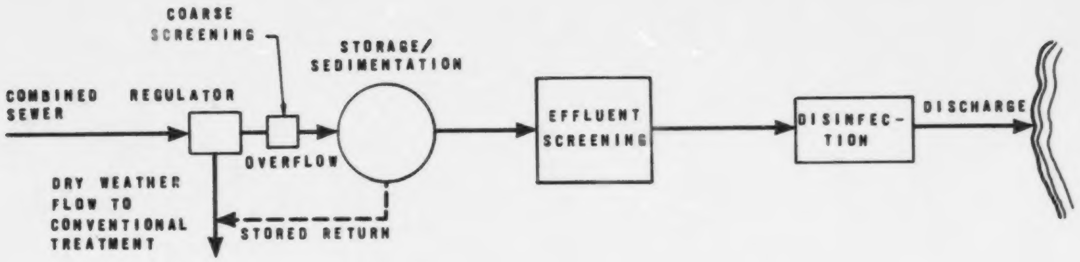

COARSE SCREENING
STORAGE/ SEDIMENTATION
COMBINED SEWER
REGULATOR
EFFLUENT SCREENING
DISINFEC-TION
DISCHARGE
OVERFLOW
DRY WEATHER FLOW TO CONVENTIONAL TREATMENT
STORED RETURN

Figure 52. Typical process flow diagram for sedimentation.

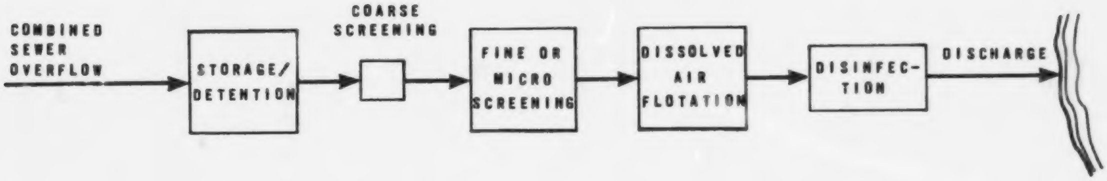

COMBINED SEWER OVERFLOW
STORAGE/ DETENTION
COARSE SCREENING
FINE OR MICRO SCREENING
DISSOLVED AIR FLOTATION
DISINFEC-TION
DISCHARGE

Figure 53. Typical process flow diagram for dissolved air flotation.

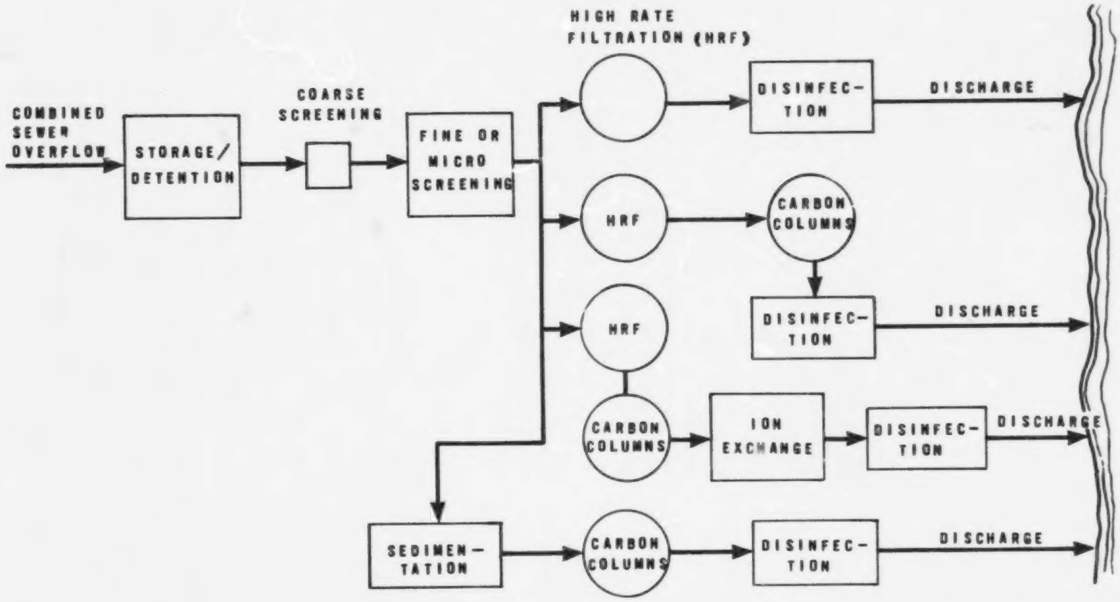

HIGH RATE FILTRATION (HRF)
COMBINED SEWER OVERFLOW
STORAGE/ DETENTION
COARSE SCREENING
FINE OR MICRO SCREENING
DISINFEC-TION
DISCHARGE
HRF
CARBON COLUMNS
DISINFEC-TION
DISCHARGE
HRF
CARBON COLUMNS
ION EXCHANGE
DISINFEC-TION
DISCHARGE
SEDIMEN-TATION
CARBON COLUMNS
DISINFEC-TION
DISCHARGE

Figure 54. Typical process flow diagram
for several advanced physical/chemical treatment systems.

TABLE 106. DESCRIPTION OF BIOLOGICAL PROCESSES USED IN COMBINED SEWER OVERFLOW TREATMENT [2, 64]

Biological process	Process description	Source of biomass	Requires additional treatment	Type of additional treatment
Contact stabilization	Process is a modified activated sludge process in which the absorption phase, or contact, and the oxidation phase (stabilization) takes place in two separate tanks. Sludge is wasted from the stabilization tank to maintain constant biomass concentrations.	From conventional activated sludge treatment facility.	Yes	Secondary clarification
Trickling filters	Standard trickling filter process in which a biological growth is supported on a stationary medium and the stormwater distributed over the surface and allowed to flow through the media. Process can include standard rate or deep bed plastic media designs.	Must be continuously maintained with a source of food.	Yes	Secondary clarification
Rotating biological contactors	Process operates on the same principle as trickling filters; however, the biological growth is supported on large diameter, closely spaced disks which are partially submerged and rotate at slow speeds.	Must be continuously maintained with a source of food.	Yes	Secondary clarification
Treatment lagoons				
Oxidation ponds	Shallow aerobic ponds which rely on surface reaeration for oxygen supply to maintain biological uptake of organics. Sedimentation also occurs in oxidation ponds.	Allowed to generate for each storm.	Usually	Algae removal by screening or filtration
Aerated lagoons	Similar to oxidation ponds except they are deeper and rely on artificial means of oxygen supply such as surface aerators or diffused air systems. System operates under aerobic conditions.	Allowed to generate for each storm.	Yes	Final clarification or sand filtration
Facultative lagoon	Facultative lagoons are the deepest of the lagoons and rely on surface reaeration. The lagoons have three distinct layers: aerobic near the surface due to algae and reaeration, a transition zone, and an anaerobic zone near the bottom sludge deposits. The biological oxidation and anaerobic stabilization occur simultaneously.	Allowed to generate for each storm.	Yes	Final clarification, screening, or sand filtration

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TABLE 107. SUMMARY OF TYPICAL BIOLOGICAL STORMWATER TREATMENT INSTALLATIONS

Project location	Type of biological treatment	Tributary area, acres	Design capacity, Mgal/d	Major process components	No. of units	Total size	Period of operation
Kenosha, Wisconsin [72]	Contact stabilization	1 200	20	Contact tank	2	32 700 ft^3	1972 to 1975
				Stabilization tank	2	97 900 ft^3	
Milwaukee, Wisconsin [71]	Rotating biological contactors	35	0.05[a]	3 ft diameter RBC units	24	28 300 ft^2	1969 to 1970
Mt. Clemens, Michigan [52]							
Demonstration system	Treatment lagoons in series with recirculation between storms	212	1.0[b]	Storage/aerated lagoon	1	750 000 ft^3	1972 to 1975
				Oxidation lagoon	1	1 100 000 ft^3	
				Aerated lagoon	1	930 000 ft^3	
Citywide full-scale system	Storage/treatment lagoons in series with recirculation between storms	1 471	4.0[b]	Aerated storage basin	1	4 440 000 ft^3	Under construction
				Aerated lagoon	1	508 000 ft^3	
				Oxidation lagoon	1	1 100 000 ft^3	
				Aerated/oxidation lagoon	1	922 000 ft^3	
New Providence, New Jersey [73]	Trickling filters	.....	6.0	High-rate plastic media	1	36 ft diameter	1970 to present
				High-rate rock media	1	65 ft diameter	
Shelbyville, Illinois [74]	Treatment lagoons:						
	Southeast site	44	28[c]	Oxidation lagoon	1	255 600 ft^3	1969 to present
	Southwest site	450	110	Detention lagoon plus 2-cell facultative lagoon	1	2 782 700 ft^3	1969 to present
Springfield, Illinois [75]	Treatment lagoon	2 208	67	Storage/oxidation lagoon	1	5 330 000 ft^3	1969 to present

a. Design based on average dry-weather flow; average wet-weather flow - 1 Mgal/d.

b. Design flowrate through lagoon systems. Total flowrate to facilities is 64 Mgal/d for the demonstration project and 260 Mgal/d for citywide system.

c. Estimated using a 50% runoff coefficient at a rainfall rate of 1.95 in./h.

acres x 0.405 = ha
Mgal/d x 0.0438 = m^3/s
ft^3 x 0.0283 = m^3
ft^2 x 0.0929 = m^2
ft x 0.305 = m
in./h x 2.54 = cm/h

Evaluation of Biological Treatment Processes

Biological treatment processes are generally categorized as secondary treatment processes, capable of removing between 70 and 95% of the BOD_5 and suspended solids from waste flows at dry-weather design flowrates and loadings. When biological treatment processes are used for stormwater treatment, removal efficiencies are lower and are controlled to a large degree by hydraulic and organic loading rates. Most biological systems are extremely susceptible to overloading conditions and shock loads as compared to physical treatment processes. However, rotating biological contactors have achieved high removals at flows 8 to 10 times dry-weather design flows [71].

Biological Treatment Performance--

Typical pollutant removals for contact stabilization, trickling filters, and RBCs are presented in Table 108, for wet-weather loading conditions. These processes include primary and final clarification. Final clarification greatly influences the overall performance of the system by preventing the carryover of biological solids produced by the processes.

TABLE 108. TYPICAL WET-WEATHER BOD AND SUSPENDED SOLIDS REMOVALS FOR BIOLOGICAL TREATMENT PROCESSES

	Expected range of pollutant removal, %	
Biological treatment process	BOD	Suspended solids
Contact stabilization	70-90	75-95
Trickling filters	65-85	65-85
Rotating biological contactors[a]	40-80	40-80

[a] Removal reflects flow ranges from 30 to 10 times dry-weather flow.

Average pollutant removal by the contact stabilization process at Kenosha, Wisconsin, is presented in Table 109. Pollutant removal effectiveness was shown to be directly dependent on the quality of the sludge being produced by the dry-weather treatment facilities. Dry-weather activated sludge is wasted to the stabilization tank to provide the biological solids when the contact stabilization system begins operation. Only after the demonstration system has operated for many hours will the sludge in the stabilization tank actually be that produced by the demonstration system and be acclimated to the waste characteristics of wet-weather flows. The dry-weather treatment plant efficiency was also improved by utilization of the demonstration project final clarifier during periods when the demonstration facilities were not in use. Dry-weather plant efficiencies increased from 82 to 94% for BOD, and from 64 to 88% for suspended solids [72].

The plastic media and conventional rock media trickling filters at New Providence, New Jersey, operate in series during dry weather, and are operated in parallel during wet weather [73]. When the system is operated in the parallel mode, overall average pollutant removal is decreased and is affected by the hydraulic flow to the plant, as shown in Figure 55. Overall pollutant removal also includes both primary and final clarification. It was also demonstrated that the plastic media filter removed about 2.7 times the BOD as compared to the rock media filter during wet-weather flows: approximately 0.86 kg BOD/m^3 as compared to 0.32 kg BOD/m^3 (54 lb/1000 ft^3 versus 20 lb/1000 ft^3) at a 45% BOD removal efficiency. A comparison of the BOD removal efficiency as a function of hydraulic and organic loading rates for the rock media and the plastic media trickling filters is shown in Figure 56.

TABLE 109. AVERAGE POLLUTANT REMOVAL PERFORMED FOR THE KENOSHA, WISCONSIN, CONTACT STABILIZATION FACILITY [72]

	Influent[a]	Effluent[b]	Removal %
Suspended solids, mg/L	299	23	90.4
Suspended volatile solids, mg/L	148	13	90.0
Total solids, mg/L	685	464	29.2
Total volatile solids, mg/L	252	130	41.6
Total BOD, mg/L	119	16	84.8
Dissolved BOD, mg/L	31	7	72.1
COD	366	66	81.9
Total organic carbon, mg/L	117	23	76.5
Dissolved organic carbon, mg/L	29	15	39.7
Kjeldahl nitrogen as N, mg/L	13.70	7.6	43.7
Total phosphate as P, mg/L	4.64	1.8	58.6
Total coliforms, MPN/mL	31 038	3 726	
Fecal coliforms, MPN/mL	2 238	443	

Note: All values indicated are arithmetic mean of 30 runs at acceptable operating levels except for coliforms which are geometric means.

a. Influent samples taken from grit tank effluent.

b. Effluent samples taken prior to chlorination.

The demonstration scale RBC at Milwaukee, Wisconsin, confirmed pilot plant results, handling a higher range of organic and hydraulic loads for periods of 8 to 10 hours [71]. A comparison of organic removal efficiency for both the pilot plant studies (using raw sewage) and the full-scale wet-weather demonstration facilities is shown in Figure 57. It was also shown that as hydraulic residence times fell below about 8 to 10 minutes, the organic removal efficiency of the demonstration facility dropped significantly. This treatment system was installed as an inline device without final clarification. Final clarifiers could greatly increase BOD and suspended solids removal by removing the sloughed biological mass caused by the high hydraulic loadings.

Lagoon Treatment Performance--

Pollutant removal efficiencies by treatment lagoons have varied from highs of 85 to 95% to negative values due to excessive algae production and carryover. In addition to the type of lagoon and the number of cells in series (stages), several major factors that influence removal efficiencies include: (1) detention time, (2) source of oxygen supply, (3) mixing, (4) organic and hydraulic loading rates, and (5) algae removal mechanisms [2, 52, 74, 75].

A single cell storage/oxidation lagoon in Springfield, Illinois, averaged 27% BOD removal and 20% suspended solids removal; however, fish kills in the receiving water were greatly reduced as compared to that prior to the construction of the facility [75]. Multiple cell facilities with algae control systems constructed at Mount Clemens, Michigan and Shelbyville, Illinois provide 75 to 90% suspended solids and BOD removal efficiencies during wet-weather conditions [52, 74].

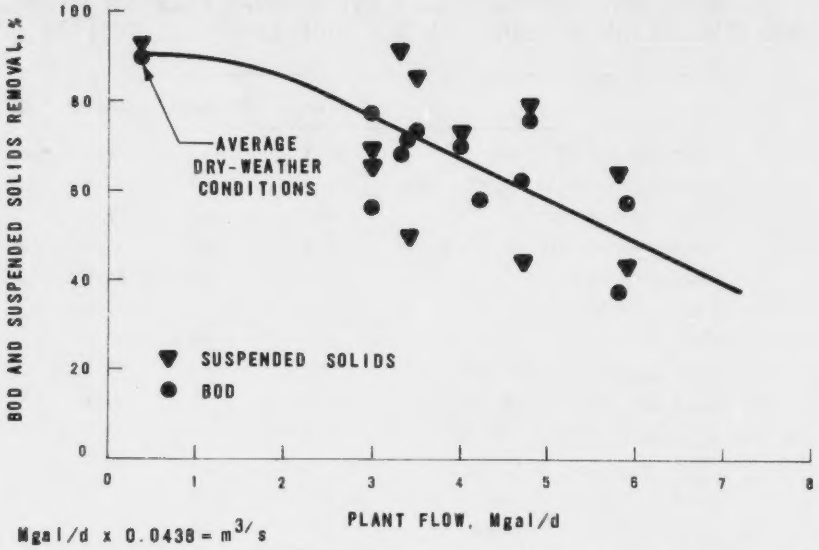


Figure 55. Overall trickling filter performance as a function of hydraulic flow, New Providence, New Jersey [73].

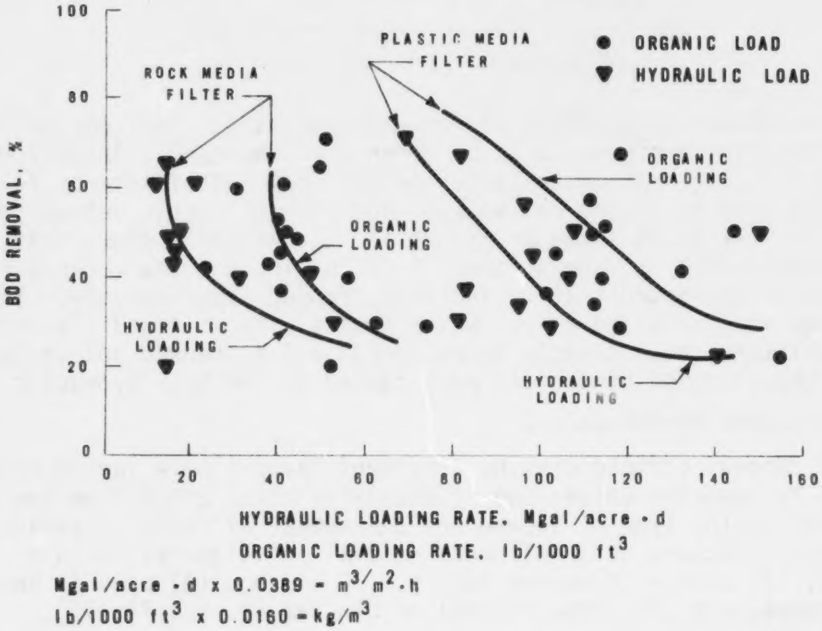
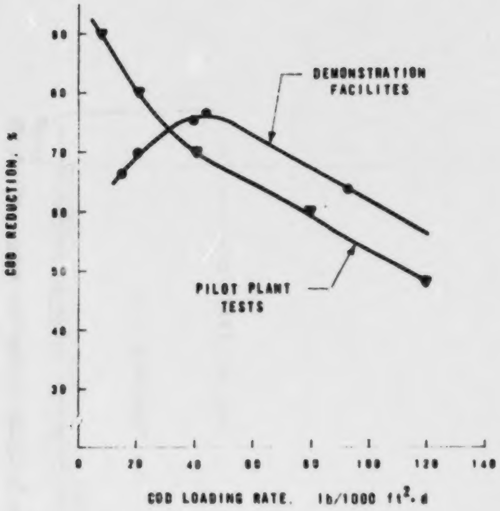


Figure 56. Comparison of rock media and plastic media trickling filters as a function of hydraulic and organic loading rates [73].



lb/1000 ft^2·d x 4.882 x 10^{-3} = kg/m^2·d

Figure 57. Comparison of COD removal performance for pilot and full scale demonstration RBC facilities, Milwaukee, Wisconsin [71].

Process efficiency profiles for suspended solids and BOD at the Mount Clemens demonstration facility are shown in Figure 58, for a 3-stage lagoon system with a microstrainer and sand filtration for suspended solids, BOD, and algae control. It was determined that intermediate algae control had little effect on the overall treatment performance [52].

Operational Problems--

An operational problem common to all stormwater biological systems is that of maintaining a viable biomass to treat flows during wet-weather conditions. For processes that borrow biomass from dry-weather facilities or allow the biomass to develop, a lag in process efficiency may be experienced as the biomass becomes acclimated to the changing waste strength and flowrate. In addition to maintaining a biological medium, clarification and/or storage are often required to provide operational control of the process, and can greatly increase capital costs of the facility.

General maintenance problems experienced by wet-weather biological facilities are similar to those experienced at conventional biological installations. Winter operation of mechanical surface aerators have had some serious drawbacks, including icing, tipping, or sinking [52, 72]. Other methods of providing the required oxygen that show promise and have been demonstrated at many dry-weather facilities include diffused air systems and submerged tube aerators [2].

At Mount Clemens, Michigan, operational problems included sludge buildup in the first cell of the lagoon system and algae control [52].

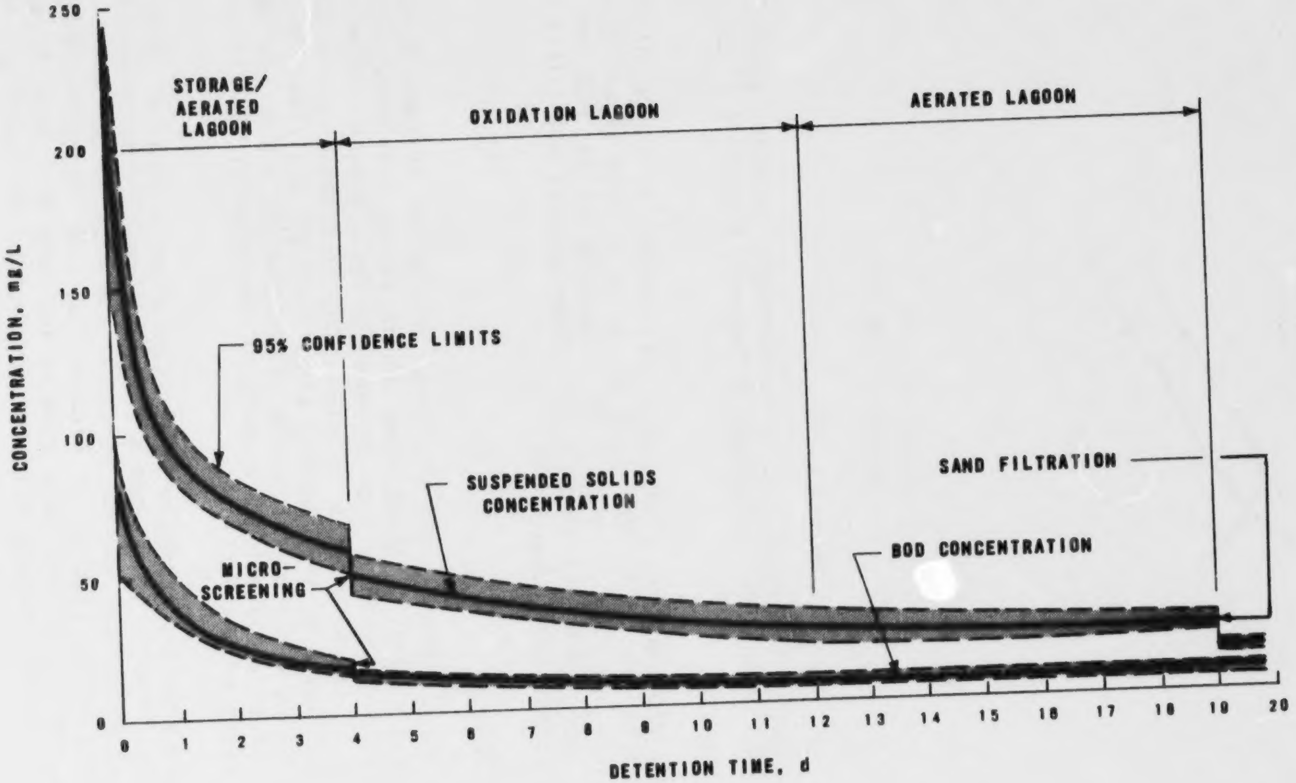


Figure 58. Suspended solids and BOD_5 concentration profiles for a 3-stage lagoon treatment system, Mount Clemens, Michigan [52].

Design Criteria--

The principal design criteria used to evaluate and design biological systems generally include hydraulic and organic loading rates, sludge and hydraulic detention times, and in the case of contact stabilization, such factors as F/M ratio, mass of organisms in the system, and rate of substrate utilization.

At Kenosha, Wisconsin, several process criteria were correlated with effluent BOD and suspended solids concentrations and removal efficiencies. The results of this correlation are presented in Table 110 [72]. These tests also indicated that low MLSS concentration of less than 2100 mg/L and high reaeration times of greater than 4 hours and long stabilization periods may seriously affect process efficiency. A contact time of at least 10 minutes was also found for satisfactory operation and performance of the facilities.

TABLE 110. RESULTS OF CORRELATION OF CONTACT STABILIZATION PROCESS PERFORMANCE AND PROCESS PARAMETERS AT KENOSHA, WISCONSIN [72]

Process equation		Multiple correlation coefficient
Effluent BOD concentration, mg/L	= 1.6 (A) + 0.92 (B) + 9.1	0.670
Effluent SS concentration, mg/L	= 2.43 (C) + 1.83 (A) + 13.9	0.544
BOD removal, %	= 0.081 (D) - 1.0 (B) - 1.3 (A) + 80.6	0.745
SS removal, %	= 0.02 (E) - 0.97 (C) - 0.7 (A) + 87.1	0.691

Note: A = F/M ratio
B = Stabilization time, d
C = Reaeration time, h
D = Influent BOD concentration, mg/L
E = Influent SS concentration, mg/L

Typical design criteria for biological treatment systems have been previously presented and discussed in the literature [2] and are summarized in Tables 111 through 114. Design criteria for treatment lagoons are not based on biological kinetic theory, but rather on actual practice and experience. An inventory and operational data from municipal lagoon facilities have been collected for various types of lagoons for each region in the United States [76]. Factors affecting lagoon performance, including organic and hydraulic loading, odor and aesthetic failures, wind, light, and mixing, are evaluated.

TABLE 111. OPERATIONAL AND DESIGN PARAMETERS FOR THE CONTACT STABILIZATION FACILITY AT KENOSHA, WISCONSIN [72]

Parameter	Average value[a]	Range of values tested[b]
MLSS concentration, mg/L	3 400	1 000-5 600
F/M ratio in contact tank, lb BOD_5/lb MLSS·d	2.8	0.5-5.0
Sludge retention time, d	2.3	0-7.0
BOD loading rate, lb BOD_5/1 000 ft^3·d	500	200-1 000
Detention time, h		
Contact tank	0.25	0.17-0.33
Reaeration time	3.0	1.0-10.0
Recycle ratio, Qr/Q	0.40	0.20-0.60
Volume of air supplied in contact tank, ft^3/lb BOD_5	250	100-700

a. Based on 30 optimized runs.

b. Ranges based on 49 runs.

lb BOD_5/lb MLSS·d = kg BOD_5/kg MLSS·d
lb BOD_5/1 000 ft^3·d x 0.016 = kg BOD_5/m^3·d
ft^3/lb BOD_5 x 62.4 = L/kg BOD_5

TABLE 112. DESIGN CRITERIA FOR TRICKLING FILTERS OPERATED IN PARALLEL FOR CONTROL OF WET-WEATHER FLOWS [2, 73]

Parameter	High rate	Ultra-high rate
Filter media	Rock	Plastic[a]
Hydraulic loading rate, Mgal/acre·d		
Recommended design	20	70
Range	10-40	40-120
Organic loading rate, lb BOD_5/1 000 ft^3		
Recommended design	40	85
Range	20-115	45-230
Depth, ft	3-8	20-40[b]
Recirculation ratio, Qr:Q	1:1-4:1	1:1-4:1

a. Or redwood slats.

b. Ultra-high rate trickling filter depth at New Providence, New Jersey = 14.4 ft.

Mgal/acre·d x 0.039 = m^3/m^2·h
lb BOD_5/1 000 ft^3·d x 0.016 = kg BOD_5/m^3·d
ft x 0.305 = m

TABLE 113. COMPARISON OF DRY-WEATHER AND WET-WEATHER DESIGN PARAMETERS FOR ROTATING BIOLOGICAL CONTACTORS

Parameter	Range of general dry-weather values [2]	Milwaukee, Wisconsin [71] Dry-weather design	Milwaukee, Wisconsin [71] Wet-weather range
Hydraulic loading rate, $gal/ft^2 \cdot d$[a]	2-8	1.8	25-35
Organic loading rate, lb BOD_5/1 000 $ft^2 \cdot d$	5-15	5.4[b]	30-70[b]
Detention time, min	15-20	69	10-20

a. Based on disc surface area.

b. Based on correlation of COD:BOD ratios.

$gal/ft^2 \cdot d \times 1.698 = L/m^2 \cdot h$
lb $BOD_5/1000\ ft^2 \cdot d \times 4.882 \times 10^{-3}$ = kg $BOD_5/m^2 \cdot d$

TABLE 114. COMPARISON OF DESIGN CRITERIA FOR TREATMENT LAGOONS [2]

	Oxidation lagoons	Aerated lagoons: Aerated oxidation lagoon	Aerated lagoons: Complete mix aerated lagoon	Facultative lagoons
Organic loading rate, lb BOD_5/acre·d	20-50	100-500	500-1 000	15-80
No. of lagoons	2-6	2-6	1-4	2-10
Depth, ft	2-5	6-10	10-15	6-12
Detention time, d	30-160	5-11	1-8	7-120[a]

a. Use of mechanical surface aerators reduces detentions to approximately 7-10 days.

lb BOD_5/acre·d x 1.1208 = kg BOD_5/ha·d
ft x 0.305 = m

Costs of Biological Treatment Facilities--

A comparison of construction, and operation and maintenance costs for biological treatment systems and treatment lagoons is presented in Table 115. Costs of final clarification are included where control of solids and sludge produced by the biological treatment system are required. Costs also include pumping, disinfection, and algae control systems when applicable. Engineering, administration, and land costs are not included in the estimates; however, land costs may be the controlling economic factor in the evaluation of lagoon treatment systems and therefore must be evaluated for each specific locations.

Many biological treatment systems are integrated with or are a part of dry-weather treatment facilities. Cost estimates of the wet-weather portion of these facilities were separated from total costs of the total treatment

TABLE 115. SUMMARY OF CAPITAL AND OPERATION AND MAINTENANCE COSTS FOR BIOLOGICAL TREATMENT ALTERNATIVES[a]

Project location	Type of biological treatment	Peak plant capacity, Mgal/d	Construction cost, $	Cost/ capacity, $/Mgal·d	Cost/ tributary area, $/acre	Annual operation and maintenance cost, ¢/1 000 gal (except as noted)
Kenosha, Wisconsin [72]	Contact stabilization	20	1 364 000	68 200	1 140	13.8
Milwaukee, Wisconsin [2, 71][b]	Rotating biological contactor	4.3	299 000	69 200	8 540	4.4
Mount Clemens, Michigan [52]						
Demonstration system	Aerated treatment lagoons	64	642 700	10 000	3 030	20.0
Citywide system	Storage/aerated treatment lagoons	260	5 737 000	22 000	3 900	19.0
New Providence, New Jersey [2, 73][c]	High-rate trickling filter	6	475 000	79 150	.....	12.3
Shelbyville, Illinois [2, 74]						
Southeast site	Oxidation lagoon	28	43 400	1 550	1 000	$1 530/yr[d]
Southwest site	Storage and facultative lagoons	110	337 700	3 070	750	$5 780/yr[d]
Springfield, Illinois [2, 75]	Oxidation lagoon	67	176 000	2 600	80	$2 100/yr

a. ENR 2000.

b. Includes estimate of final clarifier.

c. Includes plastic media trickling filter, final clarifier, plus one-half of other costs.

d. Based on estimated man-day labor requirements.

Mgal/d x 0.0438 = m^3/s
acres x 0.405 = ha
¢/1 000 gal x 0.264 = ¢/m^3

systems. The cost of the inline RBC at Milwaukee, Wisconsin, was used together with an estimated cost for a final clarifier to develop an estimated cost of a complete RBC treatment system [71]. The final clarifier cost was based on one 19.8 m (65 ft) diameter clarifier with a surface loading rate of 2.04 $m^3/m^2 \cdot h$ (1200 gal/$ft^2 \cdot d$).

Costs of lagoon treatment systems vary widely, and are a function of the type of lagoon (oxidation, aerated, or facultative); the number of cells; and the miscellaneous equipment requirements including: aeration equipment, disinfection equipment, instrumentation, pumping, and algae control provisions.

Costs for many of these stormwater facilities are based on only one installation of each biological treatment process. Therefore, these costs should be considered only coarse estimates and may be greatly influenced by the degree of integration with dry-weather treatment required to produce a viable system. These costs can be used as a preliminary guide, but detailed analysis should be performed to compare and evaluate biological treatment alternatives with other methods of treatment and control.

Biological Treatment Systems

Both single purpose and dual use (integrated biological treatment) facilities have been demonstrated in controlling combined sewer overflows. Single purpose facilities treat flows only during wet-weather conditions as in the case of the contact stabilization installation and several lagoon installations [52, 72, 75]. However, the clarifier of the contact stabilization facilities is also used for dry weather final clarification [72]. Dual use or integrated facilities are capable of treating both dry- and wet-weather flows.

Dual use has been accomplished by changing modes of operation during wet weather as demonstrated at New Providence, New Jersey. Increased performance during dry-weather was also obtained by using the trickling filters in series [73]. Biological systems have also been used to treat dry- and wet-weather flows without process modification by pushing the system to design limits as hydraulic and pollutant loads increase. Examples include the inline RBC unit at Milwaukee, Wisconsin, and the Southwest lagoon treatment system at Shelbyville, Illinois [71, 74]. At Ft. Wayne, Indiana, an existing terminal lagoon is used by both the dry-weather treatment facilities and the wet-weather screening installation prior to discharge to the receiving water [50].

Because of the limited ability of biological systems to handle fluctuating and high hydraulic shock loads, storage/detention facilities preceding the biological processes may be required. Storage/detention will be used at the citywide lagoon treatment facilities under construction at Mount Clemens, Michigan [52]. The storage unit will reduce the maximum flows entering the system from 11.39 m^3/s (260 Mgal/d) to a design flowrate of approximately 0.18 m^3/s (4.0 Mgal/d) through the lagoon system. A similar concept is also used at the Southwest treatment site in Shelbyville, Illinois [74].

Initial capital investments of integrated or dual use facilities can be reduced by apportioning part of the costs to the dry-weather facility. The cost reduction is in proportion to the net benefit that the wet-weather facility provides to the overall treatment efficiency during dry-weather periods. A description of this evaluation is presented in Section 4.

LAND TREATMENT OF STORMWATER

Land treatment methods have been used successfully to treat municipal and some kinds of industrial wastes for several years. The use of land treatment in treating wastewater or stormwater is usually limited by hydraulic application rates and the resulting land area requirements. Since stormwater volumes can be many times larger than dry-weather municipal wastewater flows, application rates are proportionally more critical in determining the economic feasibility of their application to stormwater treatment. Unless adequate flow equalization could be provided, slow rate land treatment processes with low application rates would require excessive land area.

Process Description and Facilities Installations

Based on the limitations of application rates and land area only, the following land application processes appear to have promise for treating stormwater runoff:

- Wetlands
- Rapid infiltration
- Overland flow

These methods should have application for stormwater treatment despite the absence of conclusive design, operating, and performance data from operational projects.

Wetlands--

Wetlands are areas with too many plants and too little water to be called lakes, yet they have enough water to prevent most agricultural or silvicultural uses. Existing wetlands areas are generally large enough to accommodate expected stormwater runoff volumes and their ability to influence stormwater quality appears to hold promise.

The Wayzata, Minnesota [77], project is one of the few projects currently investigating the potential of wetlands treatment, but any conclusions regarding expected quality will require more data. However, results from wetlands projects researching the potential for renovating municipal wastewater indicate effective treatment does take place [78]. The management technique for nutrient removal, loading rates, and the suitable site characteristics need further study. Winter application in northern latitudes may not be feasible.

Rapid Infiltration--

In rapid infiltration, most of the applied wastewater percolates through the soil, eventually reaching the groundwater. Rapidly permeable soils such as sands and loamy sand are suited to this process. The high application rates preclude consumptive use by plants (vegetative covers are not normally used) and there is little evaporation. Return of renovated water to the surface by wells, underdrains, or groundwater interception may be necessary or may be an advantage depending on existing groundwater quality reuse potential or water rights considerations. Rapid infiltration is only affected by the most severe climatic conditions and will require a relatively small amount of land if soil conditions are correct. Surface clogging due to high suspended solids loading can reduce infiltration rates and may require pretreatment.

Overland Flow--

In overland flow treatment, water flows across a vegetative surface to runoff collection ditches for reuse or discharge to surface water. Treatment is by physical, chemical, and biological means as a thin film of water flows over the relatively impermeable surface; very little percolation takes place.

Land Application Projects--

The only actual stormwater land treatment projects discovered in the literature are a pilot scale wetlands treatment system in Wayzata, Minnesota [77], and an experimental scale project in Tucson, Arizona [79], which combined the rapid infiltration and overland flow methods. Features of these projects are shown in Table 116.

TABLE 116. DESCRIPTION OF STORMWATER TREATMENT PROJECTS USING LAND TREATMENT

Item	Wayzata, Minnesota [77]	Tucson, Arizona [79]
Type of treatment	Wetlands	Overland flow, rapid infiltration
Hydraulic loading, Mgal/acre·yr	~2.4[a]	140-880
Land area, acres	7.5	0.02
Period of operation	November 1974 to present	Four trials, fall 1971
Preapplication treatment	Gravel roughing filter	None
Vegetative cover	Marsh vegetation	Turf grass
Surface influent and effluent monitoring	Yes	Yes
Groundwater monitoring	Observation wells and lysimeter pans	Subsurface flow collected by underdrain for monitoring
Management techniques	Intermittent application, dewatering, recirculation, and comparison with unmanaged control marsh	Four separate trials monitored changes in surface and subsurface outflow quality with respect to time

a. Hydraulic loading includes surface runoff (1.12), precipitation (0.83), and groundwater infiltration (0.42).

Mgal/acre·yr x 9 353.6 = m^3/ha·yr
acre x 0.405 = ha

These projects indicate that significant renovation is taking place, but more data are needed to support any conclusions on expected quality of the treated stormwater, pretreatment requirements, marsh fill-in, vegetation maintenance and control, and associated costs.

Evaluation of Land Treatment Alternatives

Although limited data have been compiled, an evaluation of the various land treatment alternatives using available data from stormwater treatment projects and municipal dry-weather flow projects is presented for pollutant removal efficiencies, design criteria, and costs.

Process Evaluation--

Results comparing treatment of domestic wastewater by natural and artificial marshes indicated that significant pollutant removals take place in each case [80]. It was determined that artificial marshes acted similar to natural marshes, but treatment efficiency was better for managed artificial systems. Removals were related to detention time and the length of marsh through which the wastewater passed. Treatment efficiency was adversely affected by climatic conditions; poor pollutant removals associated with the first heavy frost of the fall were observed. The best seasonal removals averaged approximately 29% for BOD and 13% for phosphorus for natural marshes. The managed artificial marsh averaged approximately 90% for BOD and 64% for phosphorus. Marsh systems can handle the high solids loading associated with stormwater runoff, and management techniques to increase pollutant removals are available.

Studies using marsh systems for stormwater treatment also indicate significant pollutant removals, as summarized in Table 117.

TABLE 117. TYPICAL POLLUTANT LOADING AND REMOVAL RESULTS USING LAND TREATMENT

	Wayzata, Minnesota [77]		
	Pollutant loading, lb/acre·yr	Removal, %	Tucson, Arizona [79]
Suspended solids	4,973	94	Results indicated significant pollutant removal, but loading and percent removals were not determined.
Phosphorus	17.8	78	
Ammonia-nitrogen[a]	64.8	0	

a. Ammonia concentrations in groundwater are higher than the stormwater influent.

lb/acre·yr x 1.121 = kg/ha·yr

Limited studies using stormwater runoff and rapid infiltration indicate good treatment performance, however, actual percent removals were not determined [79]. Several conclusions can be made from results using sanitary wastewaters:

- Pollutant removals by the filtering and straining action of the soil are excellent.
- Suspended solids, BOD, and fecal coliforms are almost completely removed.
- Nitrogen removals are generally poor unless specific operating procedures are established to maximize denitrification.

Total nitrogen removals range from 30%, without denitrification procedures, to 50% if steps to maximize denitrification are taken. Phosphorus removals can range from 70 to 90% depending on the physical and chemical characteristics of the soil.

Overland flow systems can achieve treatment to secondary level (or better) from raw, primary and treated, or lagoon treated municipal wastewater. Nitrogen and BOD removals are comparable to conventional advanced wastewater treatment. Nitrogen removals usually range from 75 to 90% with runoff nitrogen being mostly in the nitrate form. Nitrogen removal can be affected by cold weather as a result of decreased plant uptake and reduced biological activity. Phosphorus removals by adsorption and precipitation are limited because of incomplete contact between the wastewater and the adsorption sites within the soil; removals usually range from 30 to 60% on a concentration basis.

Design Criteria--

Applying alternative land treatment methods to stormwater treatment will be affected to different degrees by climatic restrictions, constituent and hydraulic loading to the system (i.e., preapplication treatment), site characteristics, and vegetative cover. Typical design features for the various processes, based on treatment of municipal wastewater, are compared in Table 118. The major site characteristics are compared for each land treatment process in Table 119.

The nitrogen, phosphorus, suspended solids, and BOD loading capacity will vary for each land treatment process depending on such factors as preapplication treatment, expected treatment performance, hydraulic limitation of the soil and underlying geology, nitrogen removal capacity of the soil-vegetation complex, and discharge standards.

For rapid infiltration systems, the infiltration capacity of the soil could be limited by excessive suspended solids loadings. If rapid infiltration is used, it is recommended that stormwater suspended solids concentrations be consistent with that of primary treated municipal effluent before application to the land. Nitrogen loading is often the limiting criterion for percolating water from rapid infiltration systems to meet EPA drinking water standards of 10 mg/L for nitrate-nitrogen. Crop uptake of nitrogen, denitrification, and storage in the soil will all affect the maximum allowable loading. Other loading parameters may include phosphorus and heavy metals.

For overland flow systems, treatment performance is directly related to pollutant loadings and hydraulic application rates. The general pollutant loading capacity depends primarily on the expected treatment performance and the level of preapplication treatment. Suspended solids reductions to a level consistent with municipal wastewater that has been screened and possibly degritted and degreased would be desirable to ensure successful operation of the system. Methods for distribution of stormwater runoff with high suspended solids loads will require careful consideration. Because application rates partially govern the expected effluent quality, maximum allowable application rates during precipitation may be relatively low. As a result, significant storage may be required affecting the economic feasibility of this process.

TABLE 118. COMPARISON OF DESIGN FEATURES FOR LAND TREATMENT PROCESSES [78]

	Application process		
Feature	Wetlands	Rapid infiltration	Overland flow
Application techniques	Sprinkler or surface	Usually surface	Sprinkler or surface
Annual application rate, ft/yr	4 to 100	20 to 560	10 to 70
Field area required, acres[a]	11 to 280	2 to 56	16 to 110
Typical weekly application rate, in./wk	1 to 25	4 to 120	2.5 to 16
Minimum preapplication treatment provided in United States	Primary treatment or coarse filtration	Primary sedimentation	Screening and grit removal
Disposition of applied wastewater	Evapotranspiration, percolation, and runoff	Mainly percolation	Surface runoff and evapotranspiration with some percolation
Need for vegetation	Required	Optional	Required

a. Field area in acres not including buffer area, roads, or ditches for a 1 Mgal/d (43.8 L/s) flow.

ft/yr x 0.3048 = m/yr
acres x 0.405 = ha
in./wk x 2.54 = cm/wk

TABLE 119. COMPARISON OF SITE CHARACTERISTICS FOR LAND TREATMENT PROCESSES [78]

	Application process		
Characteristics	Wetlands	Rapid infiltration	Overland flow
Slope	Usually less than 5%	Not critical; excessive slopes require much earthwork	Finish slopes 2 to 8%
Soil permeability	Slow to moderate	Rapid (sands, loamy sands)	Slow (clays, silts, and soils with impermeable barriers)
Depth to groundwater	Not critical (zero)	10 ft (lesser depths are acceptable where underdrainage is provided)	Not critical
Climatic restrictions	Storage may be needed for cold weather	None (possibly modify operation in cold weather)	Storage often needed for cold weather

ft x 0.3048 = m

Costs of Land Treatment Systems--

There is an absence of full scale operational projects where capital and operating costs have been compiled. However, cost curves for rapid infiltration and overland flow systems which treat municipal wastewater have been compiled presenting component capital and operating costs [81].

The use of existing wetlands already influenced by stormwater would appear to be very economical but existing sites are not always available. Creation of artificial wetlands is another approach which has received some attention as a low cost land treatment method.

DISINFECTION

Disinfection of storm and combined sewer overflows is generally practiced at all stormwater treatment facilities to control pathogens and other microorganisms in receiving waters. At most stormwater installations, disinfection has been accomplished by applying conventional wastewater technology supplemented by high rate processes and on-site generation of disinfectant. Several aspects of disinfection practices require reconsideration for stormwater treatment applications. These include:

- A residual disinfecting capability may not be feasible for stormwater discharges. Recent work indicates that chlorine residuals and compounds discharged to natural waters may be harmful to aquatic life.

- The coliform count is increased by surface runoff in quantities unrelated to pathogenic organism concentration. Total coliform levels may not be the most useful indication of disinfection requirements and efficiencies.

- Discharge points requiring disinfection are often at outlying points on the sewer system and require unmanned, automated installations.

- Storm flow is highly variable both in quantity and quality; disinfection facilities must be able to meet these fluctuations.

Three basic needs for control of microorganisms in stormwater overflows have been identified [82]: (1) to obtain knowledge of the storm flow's microorganism pathogenic quality and the pathogens' relationships to other indicator organisms; (2) to develop high-rate disinfection systems to reduce large tankage and/or dosage requirements, and (3) to develop disinfection facility design and operation techniques for the highly varying quality and quantity characteristics of storm flows.

Disinfection Projects

Demonstration projects evaluating stormwater disinfection technology are summarized in Table 120. Other projects, evaluating the characteristics and impacts of microorganisms in stormwater, have been beneficial in providing a background understanding of the sources and constituents of microbial contamination in overflows [82-85].

TABLE 120. SUMMARY OF DEMONSTRATION STORMWATER DISINFECTION PROJECTS

Project location	Disinfectant agent	Source	Description of disinfection system	Period of operation
Boston, Massachusetts [17]				
Cottage Farm Detention and Chlorination Station	Sodium hypochlorite (NaOCl)	Purchased/ stored	Automatic disinfection system injects up to 3 000 gal of 10 to 15% NaOCl into the influent channel to the detention basins for the design storm.	1971 to present
Cleveland, Ohio [86]	Sodium hypochlorite (NaOCl)	Purchased/ stored	Disinfection of two bathing beaches enclosed by fabric barriers and disinfection of polluted streams and overflow points influent to Lake Erie.	1968 to 1970
Fitchburg, Massachusetts [87]	Sodium hypochlorite (NaOCl)	Purchased/ stored	High-rate application of disinfectant via thin film in a Dynactor. System incorporates chemically assisted high-rate settling.	1974 to present
New Orleans, Louisiana [88]	Sodium hypochlorite (NaOCl)	Central generation	NaOCl is generated at a central manufacturing facility with a capacity of 1 000 gal/h. The 12% NaOCl is transported and stored at 4 pumping stations on 3 overflow channels to disinfect pumped stormwater.	1972 to present
New York City, New York [25]				
Spring Creek	Sodium hypochlorite (NaOCl)	Purchased/ stored	Automatic disinfection system injects up to 60 000 lb/d of 5% NaOCl into the inlet sewer of the storage/ detention facilities.	1972 to present
Philadelphia, Pennsylvania [55, 56, 57]	Sodium hypochlorite (NaOCl) Ozone (O_3)	Purchased On-site generation	Comparison of two disinfectants on screened and unscreened combined sewer overflow. Short contact times are achieved by high velocity gradients in a plug flow contact chamber regime.	1969 to 1973
Rochester, New York [36]	Chlorine (Cl_2) Chlorine dioxide (ClO_2)	Purchased On-site generation	Sequential addition of Cl_2 and ClO_2 with flash mixing at each point of application. Disinfection is final treatment step following sedimentation, storage, dual media filtration, and carbon column pilot facilities.	1975 to 1976
Syracuse, New York [35, 89]	Chlorine gas (Cl_2) Chlorine dioxide (ClO_2)	Purchased On-site generation	Evaluation of individual and sequential addition of Cl_2 and ClO_2 following treatment of combined sewer overflows by screening and swirl concentration.	1974 to present

gal x 3.785 = L
lb/d x 0.454 = kg

The Fitchburg, Massachusetts, demonstration facility represents a new technology in disinfectant application [87, 90]. The 373 m^3/d (100 000 gal/d) combined sewer treatment facility includes chemical addition ($FeCl_3$, CaO, and polymer) and high-rate settling prior to disinfection.

Disinfection is accomplished by the use of thin film technology. Hypochlorite is sprayed on a thin film of wastewater to provide maximum instantaneous contact and eliminate the need for further mixing. A small sump is provided at the outlet of the unit but no contact chamber is required. Analysis indicates that both total coliform and fecal coliform are reduced to less than 36 organisms per 100 mL.

A second high-rate settling unit after disinfection was found to add little to the overall suspended solids removal efficiency. Typical pollutant removals for the facility average 65% for BOD and COD, 85% for suspended solids, 90% for total phosphorus, and over 99% for total and fecal coliforms.

Future studies proposed at Fitchburg will include the use of ozone as a disinfecting agent.

Disinfection Agents

The disinfection agents used in wastewater and stormwater treatment include chlorine, calcium and sodium hypochlorite, chlorine dioxide, and ozone. Results from combined bench and pilot plant testing of high gradient magnetic separators indicate 99.9% removal of viruses and over 99% removal of total and fecal coliforms [28]. However, physical methods and other chemical agents have not experienced wide usage either because of excessive costs or difficulties with application technology.

Evaluation of Disinfection Agents--

The four potential disinfection agents have some common characteristics; all are oxidizing agents, corrosive to equipment, and are highly toxic to both microorganisms and higher life. Other characteristics and differences that should be considered when choosing a stormwater disinfectant are summarized in Table 121. A discussion of these characteristics follows.

Stability--The more stable chemicals allow the designer greater flexibility in developing a treatment facility. Chlorine gas is always purchased and its high degree of stability allows long storage periods. Hypochlorite can be purchased or generated onsite and can be stored for several months, or it can be generated at a steady rate and stored between overflow events. Peak demand requirements can come from storage or be purchased as needed.

At New York's Spring Creek facility, purchased sodium hypochlorite is diluted and stored at a strength of about 5% available chlorine, which reduces the rate of deterioration [25]. It has been shown that the stability of sodium hypochlorite is higher at reduced concentrations [2]. Chlorine dioxide and ozone are the least flexible; they must be generated onsite and their effective lives are too short to make storage practical. Consequently, disinfectant generating capacity must be sufficient to handle anticipated peak demands.

TABLE 121. CHARACTERISTICS OF PRINCIPAL STORMWATER DISINFECTION AGENTS

Characteristic	Chlorine	Hypochlorite	Chlorine dioxide	Ozone
Stability	Stable	6 month half-life	Unstable	Unstable
Reacts with ammonia to form chloramines	Yes	Yes	No	No
Destroys phenols	At high concentrations	At high concentrations	Yes	Yes
Produces a residual	Yes	Yes	Short lived[a]	No
Affected by pH	More effective at $pH < 7.5$	More effective at $pH < 7.5$	Slightly	No
Hazards	Toxic	Slight	Toxic, explosive	Toxic

a. Chlorine dioxide dissociates rapidly [89].

Chemical Reactions--Chlorine and hypochlorite will react with ammonia to form chloramines and with phenols to form chlorophenols. These are toxic to aquatic life and the latter also produces taste and odor in the water. Chlorine dioxide does not react with ammonia and completely oxidizes phenols. Ozone is also more effective in oxidizing phenols. Elimination of possible odor and taste problems will be important for water reuse or recyle.

Residuals--Chlorine and hypochlorite disinfection have a residual that can be easily measured, will provide protection against aftergrowth, and can be used to control application rates without biological testing. Chlorine dioxide dissociates rapidly and can be measured by election spin resonance (esr), a procedure that requires more elaborate equipment than measuring common chlorine residuals and by modified DPD techniques [89]. Ozone does not have a measurable residual and would have to be controlled by biological testing.

pH Effect--Both chlorine and hypochlorite disinfection depend on the HOCl phase of the chlorine-water reaction. This phase predominates at low pH and is essentially absent above a pH of 9; therefore, these agents are limited to acidic or neutral wastewaters. A study at Fort George Meade, Maryland, showed that both coliform and virus removal was improved with pH adjustment to the acid range [91]. The effective pH range for chlorine dioxide lies between 4.5 and 7.5 [89]. It should be noted, however, that adjustment of pH below 5 may be unacceptable to prevailing stream conditions and standards, and may not be economically justifiable. Ozone is relatively unaffected by pH and can be used on any waste or at any point in the treatment scheme.

Hazards--Chlorine, chlorine dioxide, and ozone are all dangerous gases that must be carefully handled by competent personnel. The hazards of chlorine gas are well known and have caused restrictions of its use or transport in several cities including New York and Chicago. Gas concentrations as low as 5 ppm can cause difficulty in breathing and 1000 ppm can be toxic. Chlorine dioxide has toxicities similar to chlorine gas and the additional danger of exploding with any slight change in environment. It must be kept in the aqueous state to minimize dangers. The gas is soluble in water but does not react chemically with water. Ozone's oxidizing capacity makes concentrations of 1.0 ppm in the atmosphere hazardous to health. Hypochlorite can be obtained as a solid or liquid and does not have the potential dangers of the other three agents. It is the safest choice for remote, unmanned disinfection operations.

Evaluation of Application Technology--

Several studies have been conducted to examine application techniques that improve or enhance the disinfecting capability. Adequate mixing under plug flow conditions and sequential addition of chlorine (Cl_2) and chlorine dioxide (ClO_2) were two significant parameters which influenced disinfection efficiency.

Mixing--In high-rate disinfection systems where contact times are less than 10 minutes, usually in the range of 1 to 5 minutes, adequate mixing is a critical parameter, providing complete dispersion of the disinfectant and forcing disinfectant contact with the maximum number of microorganisms. The more physical collisions high-intensity mixing causes, the lower the contact time requirements. Mixing can be accomplished by mechanical flash mixers at the point of disinfectant addition and at intermittent points, or by specially designed contact chambers, or both [2, 36, 55].

At Philadelphia [55, 57], a specially designed contact chamber with closely spaced corregated baffles was used to increase the velocity gradient (G) in t^{-1}. G is a function of the viscosity of the fluid, velocity, and headloss. In this application it was considered desirable to keep the product of G and detention time (t) a constant, at less than peak design flow conditions. Assuming that t remains constant, therefore velocity remains constant, G is increased by increasing the headloss through the use of corregated channels [2]. Spacing and arrangement of the channels is also essential to maintain plug flow conditions preventing any backmixing of the dispersed disinfectant. Using this design, a contact time of 3 minutes with initial chlorine concentrations as low as 2.6 mg/L, reductions of total and fecal coliforms by 99.9% were obtained.

At an experiment at Fort George Meade to show the effect of mixing on disinfection, turbulence was created in a sewage effluent line by installing a 20.3 cm (8 in.) orifice to increase flow velocities to the range of 2 to 2.3 m/s (6.6 to 7.6 ft/s). Virus kills were increased to 83.6 to 99.3% from 45.8 to 73.5%; however, it was found that coliform kills did not substantially increase [91].

Sequential Addition of Disinfectants--Disinfection was shown to be enhanced beyond the expected additive effect by sequential addition of Cl_2 followed by

ClO_2 at intervals of 15 to 30 seconds [36, 82, 89]. A minimum effective combination of 8 mg/L of Cl_2 followed by 2 mg/L of ClO_2 was found as effective as adding 25 mg/L Cl_2 or 12 mg/L ClO_2 individually in reducing total and fecal coliforms, fecal streptococci, and viruses to acceptable target levels [82,89].

It was surmised that the presence of free Cl_2 in solution with chlorite ions (ClO_2^-), (the oxidized state of ClO_2), may cause the reduction of ClO_2^- back to its original state. This process would prolong the existence of ClO_2, the more potent disinfectant [82, 89].

Other significant findings of the Onondaga County, New York, studies include the following:

- Sequential doses of the same disinfectant do not increase disinfection over a single dose with the same total quanity.
- Prescreening does not appear to affect Cl_2 disinfection but slightly improves disinfection with ClO_2.
- Cl_2 and ClO_2 demands may be due to different materials in wastewater.
- The maximum antiviral activity of ClO_2 was found to occur between pH 4.5 and 7.5.
- Increases of temperature from 2°C to 30°C (36° to 86°F) slightly improved high-rate bacterial disinfection with both Cl_2 and ClO_2. Viral inactivation with ClO_2 was sharply decreased at 4°C (39°F) but unaffected between 12°C and 36°C (54°F and 97°F).

Aftergrowth of Microorganisms

Aftergrowth of indicator microorganisms in stormwater after disinfection have been reported [84, 86, 89]. Indicator microorganisms, specifically total coliforms, enter a log growth phase when the disinfectant residual decreases to undetectable values. Aftergrowth coliform levels can exceed before disinfection background levels. Total and fecal coliform aftergrowth were reported during stream and laboratory studies at Cleveland, Ohio [86]. Only total coliform aftergrowth was reported during a stormwater disinfection study at The Woodlands, Texas [84]. In both cases, aftergrowth of fecal streptococci did not occur. Laboratory aftergrowth studies in Syracuse, New York, revealed that difficulties in simulating the conditions for aftergrowth may be encountered for bench scale tests [89]. Aftergrowth tests, conducted to determine the ultimate bacterial and viral counts that might result in the receiving water from the discharge of untreated and disinfected combined sewer overflow, showed no measurable increases during and up to 3 days. These results were felt to be more indicative of the inability to simulate receiving water conditions in the laboratory rather than a lack of aftergrowth.

A possible chemical change in the composition of the stormwater caused by chlorine may enhance aftergrowth. This chemical change is assumed to be a cleavage of large protein molecules into smaller proteins, peptides, and amino acids. These smaller molecules are more readily available to the bacteria for growth and reproduction than the larger proteins [86].

The City of Cleveland conducted a research study to determine the cause of the aftergrowth that occurred during the hypochlorination of the streams [86]. Also, possible methods to reduce aftergrowth were investigated. The study consisted of: (1) a stream study of bacterial aftergrowth resulting from hypochlorination, and (2) bench scale studies on possible relationships between aftergrowth and chlorination due to chlorination-induced changes. Conclusions of the bacterial aftergrowth study are summarized as follows:

- Hypochlorination of streams results in a significant reduction of indicator bacteria; however, as soon as the chlorine residual dissipates, a bacterial aftergrowth occurs.

- Fecal streptococcus exhibited a very limited ability for aftergrowth in the laboratory. Fecal coliforms displayed a moderate ability for aftergrowth. Total coliforms were capable of aftergrowth that closely approximated, or exceeded, their respective initial levels.

- Factors found to significantly affect bacterial aftergrowth are: (1) the extent of dilution of the chlorinated water; (2) time available for aftergrowth between chlorination and dilution; and (3) levels of residual chlorine.

- While maintaining a 6 mg/L chlorine residual throughout a laboratory study, no significant decrease in aftergrowth was noted by increasing the chlorination detention time from 15 minutes to 72 hours.

- Proteins, as analyzed by the Lowry Method for protein determination, were greatly increased in stream water samples upon the addition of sodium hypochlorite. It is assumed that chlorine cleaves larger protein molecules into smaller proteins, peptides, and amino acids which yield more reactive sites to react with the Lowry color development reagent. All the reactive nitrogenous organic compounds were calculated as protein. Since both laboratory and field studies show bacterial populations were greater after chlorination than before, it is further hypothesized that the smaller nitrogenous compounds were more easily utilized by the bacteria for growth and reproduction which could be significant in the rate and magnitude of bacterial aftergrowth.

- Other than of proteinaceous material, there were no appreciable chlorination induced chemical-physical changes in the water samples studied that could be demonstratably related to bacterial aftergrowth.

A multipurpose investigation of surface water quality and disinfection was conducted in a 8100 ha (20 000 acre) test site at The Woodlands, Texas [84]. It was found that following disinfection of stormwater with either chlorine.

ozone, or bromine with dosing up to 32 mg/L, aftergrowth occurred after 4 to 8 days. Aftergrowth occurred only in the total coliform group.

Biological Indicator Organisms

Total coliforms, fecal coliforms, and fecal streptococci are the most common biological indicator organisms used to measure water, wastewater, and stormwater pathogenic quality and disinfection efficiency. Because extremely high coliform counts can come from natural background sources other than humans, the use of the coliform group as an indicator of the presence of pathogens in stormwater has been questioned [84, 85]. Analysis of soil samples taken from areas adjacent to established stream sampling stations and from other areas of The Woodlands, Texas, yielded positive values for all indicator bacteria groups, including pathogens [84]. In Baltimore, investigations have also revealed little or no correlations between indicator and pathogenic bacteria in storm and stream samples; however, pathogens were received in all stormwater samples [85].

In using coliform counts to measure or control disinfection efficiency, and as a basis of design when the possibility of aftergrowth of coliform organisms exists and/or potentially high background levels exist, gross over or under design of disinfection facilities may result.

Studies have been conducted to evaluate alternative microbial indicators including high chlorine resistant organisms, pathogens themselves, fecal coliforms to fecal streptoccus ratios, and adenosine triphosphate [83, 84, 89, 92].

The coliform group of indicator organisms have a relatively low chlorine resistance when compared to such pathogens as enteric viruses and protozoan cysts. Three indicators were investigated which were resistant to chlorination in the range considered necessary for the inactivation of pathogens and viruses. These included a yeast and two acid-fast bacillus [83]. Similar studies were conducted in Syracuse, New York, using bacteriophage f2 and ØX174, Polio-1, and other viruses that are more resistant to chlorination than the coliform indicator bacteria [89].

Measurement of pathogens themselves is a method to identify microbial quality directly [85, 92]. However, procedures to isolate and enumerate viruses such as *Salmonella*, *Shigella*, *Pseudomonas aeruginosa*, and *Staphlococcus aureus* are considerably more difficult than for the coliform group. Better methods and reliable qualitative recovery procedures for the enumeration of pathogenic microorganisms should be developed to identify pathogen presence and impact in storm runoff and combined sewer overflows [85].

Measurements of fecal streptococcus in addition to total and fecal coliforms may provide an indication of the source of the polluting bacteria groups through the use of the fecal coliforms/fecal streptococci ratio (FC/FS) and fecal coliform/total coliform ratio (FC/FT) [82, 84, 85]. An FC/FT ratio of greater than 0.1 is believed to be indicative of sewage; however, a firm FC/FT ratio has been difficult to establish. An FC/FS ratio of 4.0 or greater is believed to be indicative of human sources and a ratio of 1.0 or less is believed to be indicative of animal sources. The FC/FS ratios between 0.7 and

4.0 are difficult to interpret. It is suggested that FC/FS ratios be applied carefully and that the ratios are most meaningful when data are collected at discharge points to the receiving water. Upon entering receiving waters, the levels of each of the microorganisms may be affected by numerous environmental factors and differential microbial die-away [85].

In samples of storm runoff, FC/FS ratios of less than 1.0 have been noticed and FC/FS ratios representative of combined sewer flows had only 18% of samples greater than 4.0, indicating animal sources of contamination [85].

A potential alternative to microbial indicators is the use of adenosine triphosphate (ATP), a substance that is universally found in all living cells. Significant decreases in ATP that parallel bacteria reductions have been observed during the disinfection process. It may be feasible to use ATP as an instantaneous measure and a control for disinfection processes [89].

Costs of Stormwater Disinfection Systems

Costs of disinfection systems used to treat combined sewer overflows and stormwater discharges can vary greatly depending on the complexity of the system. Stormwater disinfection must be flexible and capable of automatic operation to handle intermittent and varying flows and volumes. Summaries of typical disinfection costs are presented in the literature for chlorine gas, hypochlorite, and ozone systems [2, 27].

Costs used for disinfection alternative selection should be evaluated using local conditions and requirements. These can include disinfection and receiving water requirements amd standards, equipment and disinfectant availability and costs, and system control and operation requirements.

Improvements and changes in on-site generation equipment may make these alternatives more economically attractice for storm flow applications. Ozone generation, although more expensive than other methods of disinfection, may become an economically feasible alternative in light of increasingly strict control of residuals and compounds formed by chlorine disinfection and the increasing costs of chlorine [82].

Cost curves comparing chlorine gas, chlorine dioxide, and hypochlorite generation disinfection systems have been developed and are presented in Figure 59. These costs (ENR 2000) include manufactured equipment, piping, housing, electrical and instrumentation, and miscellaneous items. No allowance for contingency or land was included. Operation and maintenance cost curves have also been developed and include annual labor requirements; miscellaneous supply costs for chlorine gas, chlorine dioxide, and hypochlorite disinfection systems; and power requirements for hypochlorite generation [27].

ILLUSTRATIVE PROBLEMS

Comparison of several stormwater treatment technologies together with examples of process design and cost evaluations are presented in Example Problems 7-1 through 7-5. The problems include a cost-effectiveness comparison of total storage and storage/sedimentation; design of a swirl concentrator, including geometry modifications; development of an equation for estimating operation and maintenance costs for storage facilities; and a method for optimizing integrated storage/treatment facilities. An evaluation of land requirements and design considerations for land treatment of stormwater is also presented.

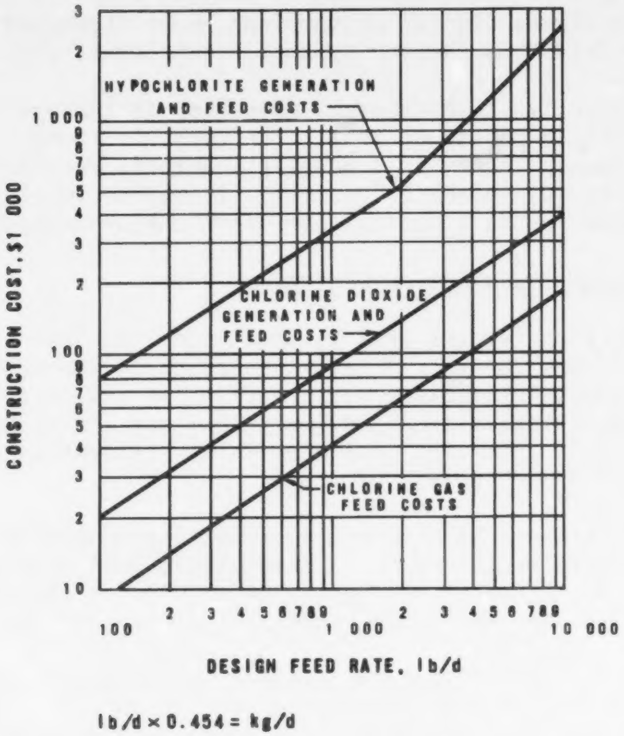


Figure 59. Chlorine disinfection cost curves, ENR 2000 [27].

EXAMPLE PROBLEM 7-1: ASSESSMENT OF STORAGE AND STORAGE/SEDIMENTATION COSTS AND COST EFFECTIVENESS

Given a frequency curve of storm rainfall, determine the costs, annual pollution reduction, and cost effectiveness for a storage and a storage/sedimentation facility. The storage facility is to be design to capture 95% of the total annual runoff volume. The storage/sedimentation facility is to be designed to capture 50% of the total annual runoff volume and treat those flows exceeding storage capacity by sedimentation.

Specified Conditions

1. Drainage area = 1000 acres.
2. Average runoff coefficient = 0.50
3. Total annual rainfall = 44 in.
4. Average suspended solids (SS) concentration in runoff = 400 mg/L
5. Construction costs (ENR 2000) for earthen-lined reservoirs = $0.25/gal of volume; for concrete sedimentation tanks, $1.00/gal of volume.

Assumptions

1. The storage volumes are based on a frequency plot of total storm rainfall, as shown in Figure 7-1.

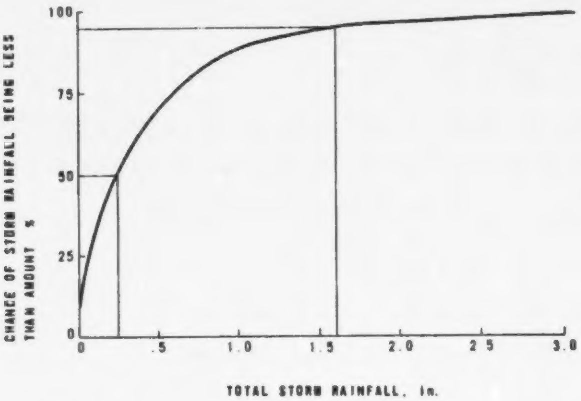


Figure 7-1. Percent chance of obtaining less than total storm rainfall amount.

2. It is assumed that runoff follows the same relationship of frequency as the rainfall curve.
3. Average flowrate to the storage/sedimentation facilities for flows exceeding storage capacity is based on the average of the maximum hourly rainfalls for each storm.
4. All flows totally contained in storage are to be released back to the interceptor and receive 85% removal at a dry-weather treatment facility.

Solution

1. Compute the volume and the construction cost for 95% storage of the annual runoff volume. From Figure 7-1, capture of storm rainfalls of less than 1.6 in. will result in a 95% capture of the annual rainfall volume.

    a. Determine design runoff amount using a 50% runoff coefficient.

    Design runoff amount = 1.6 in. x 0.50
    = 0.80 in.

b. Determine storage volume required.

$$\text{Storage volume} = \frac{(0.80 \text{ in.} \times 1000 \text{ acres})\ (43\ 560 \text{ ft}^2/\text{acre})}{12 \text{ in./ft}}$$
$$= 2.90 \times 10^6 \text{ ft}^3$$
$$\text{or } 21.7 \text{ Mgal}$$

c. Compute the construction cost of the storage facility.

$$\text{Cost} = 21.7 \times 10^6 \text{ gal} \times \$0.25/\text{gal}$$
$$= \$5\ 425\ 000$$

d. Adjust ENR 2000 costs to current costs.

$$\text{ENR } 2500 \text{ costs} = \$5\ 425\ 000 \times 1.25$$
$$= \$6\ 780\ 000$$

2. Determine the volume and the construction costs for 50% storage of the annual runoff volume for the storage/sedimentation facilities:

a. $$\text{Storage volume} = \frac{([0.24 \times 0.50] \times 1000)\ (43\ 560)}{12}$$
$$= 435\ 600 \text{ ft}^3$$
$$\text{or } 3.26 \text{ Mgal}$$

b. $$\text{Cost of storage facility} = 3.26 \times 10^6 \text{ gal} \times \$1.00/\text{gal}$$
$$= \$3\ 260\ 000$$

c. $$\text{ENR } 2500 \text{ cost} = \$3\ 260\ 000 \times 1.25$$
$$= \$4\ 075\ 000$$

3. Determine the total SS removed by the storage system capturing 95% of the annual runoff volume.

a. Compute annual runoff volume for a total annual rainfall of 44 in.

$$\text{Annual runoff volume} = \frac{([44 \times 0.50] \times 1000)\ (43\ 560)}{12}$$
$$= 79.8 \times 10^6 \text{ ft}^3/\text{yr}$$
$$\text{or } 597 \text{ Mgal/yr}$$

b. Compute annual SS load at 400 mg/L ≃ 400 ppm

$$\text{Annual load} = 597 \text{ Mgal/yr} \times 8.34 \text{ lb/gal} \times 400 \text{ ppm}$$
$$= 2 \times 10^6 \text{ lb/yr}$$

c. Compute the SS load contained in storage

$$\text{SS captured} = 2 \times 10^6 \text{ lb/yr} \times 0.95$$
$$= 1.9 \times 10^6 \text{ lb/yr}$$

d. Compute the SS removed by conventional treatment at a rate of 85%.

$$\text{SS removal} = 1.9 \times 10^6 \times 0.85$$
$$= 1.62 \times 10^6 \text{ lb/yr}$$

4. Determine the total SS removed by storage/sedimentation capturing 50% of the annual runoff volume and treating the remainder by sedimentation.

a. Compute the annual SS load contained in storage and treated at a conventional dry-weather facility achieving 85% removal.

$$\text{SS removal} = 2 \times 10^6 \text{ lb/yr} \times 0.50 \times 0.85$$
$$= 850\ 000 \text{ lb/yr}$$

b. Determine the average flowrate for flows that exceed storage capacity using an average maximum hourly rainfall of 0.20 in./h.

$$\text{Runoff rate} = \frac{(0.20 \text{ in./h} \times 0.50)\ (1000 \text{ acres})\ (43\ 560 \text{ ft}^2/\text{acre})\ (7.48 \text{ gal/ft}^3)\ (24 \text{ h/d})}{12 \text{ in./ft}}$$
$$= 65.2 \text{ Mgal/d}$$

c. Determine surface area of the storage/sedimentation basin at a 10 ft sidewater depth (swd).

$$\text{Area} = \frac{435\ 600\ \text{ft}^3}{10\ \text{ft}} = 43\ 560\ \text{ft}^2$$

(see 2.a.)

d. Compute average hydraulic loading rate.

$$\text{Hydraulic loading rate} = \frac{65.2 \times 10^6\ \text{gal/d}}{43\ 560\ \text{ft}^2} = 1500\ \text{gal/ft}^2\cdot\text{d}$$

e. Determine the average SS removed by sedimentation at a hydraulic loading rate of 1500 $\text{gal/ft}^2\cdot\text{d}$. Using Figure 38, SS removal = 30%.

$$\text{SS removed by sedimentation} = 2 \times 10^6\ \text{lb/yr} \times 0.50 \times 0.30 = 300\ 000\ \text{lb/yr}$$

f. The total SS removed by the storage/sedimentation facilities is 850 000 + 300 000 = 1.15×10^6 lb/yr.

5. Estimate the annual costs, including various land costs for storage and storage/sedimentation. Also determine the cost effectiveness for each type of storage.

a. Determine the gross land area requirements for storage, using a 10 ft swd and the typical section of an earthen embankment as shown in Figure 7-2.

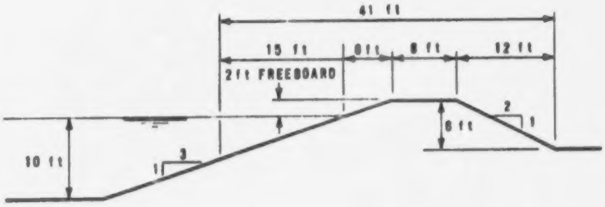


Figure 7-2. Typical earthen embankment detail.

$$\text{Effective water surface area} = \frac{2.9 \times 10^6\ \text{ft}^3}{10\ \text{ft}} = 290\ 000\ \text{ft}^2$$

(see 1.b.)

or 538.5 ft x 538.5 ft

$$\text{Gross area} = (538.5 + [2 \times 41])^2 = 385\ 000\ \text{ft}^2$$

or 8.84 acres

b. The area required for storage/sedimentation = $\frac{43\ 560\ \text{ft}^2}{43\ 560\ \text{ft}^2/\text{acre}}$ = 1.0 acre
No additional area is required because of the vertical concrete walls.

c. Estimate the land cost for storage at $10 000/acre.

Land cost = 8.84 acres x $10 000/acre
= $88 400

d. Compute the total construction and land cost for storage.

Cost = $6,780 000 + $88 400
= $6 868 400

e. Compute the amortized construction costs for storage using a 20 yr life at 7% interest.

Amortized construction cost = total cost x capital recovery factor (20,7)
= \$6 868 400 x 0.09439
= 648 000

f. The amortized construction costs for both storage and storage/sedimentation using land costs of \$10 000/acre, \$25 000/acre, and \$50 000/acre are summarized as follows:

Amortized construction costs, \$/yr

	Land costs, \$/acre		
	10 000	25 000	50 000
Storage	648 000	661 000	682 000
Storage/sedimentation	386 000	387 000	389 000

g. Determine the cost effectiveness using amortized construction costs together with the total pounds of SS removed per year for the two types of storage at each land cost. The cost effectiveness for storage at \$10 000/acre $= \frac{\$648\ 000/\text{yr}}{1.62 \times 10^6\ \text{lb/yr}}$
$= \$0.40/\text{lb}$

Cost effectiveness values for all determinations are summarized as follows:

Cost/lb SS removed, \$/lb

	Land costs, \$/acre		
	10 000	25 000	50 000
Storage	0.40	0.41	0.42
Storage/sedimentation	0.34	0.34	0.34

Comment

Although actual construction and land costs will vary from the values in this example, it can be seen that land costs affect storage costs and cost effectiveness to a greater degree than storage/sedimentation. A higher percentage of large total rainfall would require even larger storage facilities; however, if the majority of total rainfall volumes were small, total storage may approach the most economical and cost-effective solution.

EXAMPLE PROBLEM 7-2: DESIGN OF A SWIRL CONCENTRATOR/REGULATOR

Using the design curves developed from model studies [29], determine the design details for a swirl concentrator/flow regulator removing 90% settleable solids, and indicate the range of removals over the range of influent flows. Also, develop revised design dimensions using a weir height (H) equal to the inlet dimensions (D_1).

Specified Conditions

1. The design flow = 40 ft^3/s
2. The influent sewer size = 3 ft

Assumptions

1. The peak flow = 90 ft^3/s

Solution

1. Determine the standard design details ($H_1/D_2 = 0.25$) for the swirl concentrator/regulator.

   a. From Figure 7-3 (Figure 7 in reference [29]), determine the chamber diameter (D_2) for a design flow of 40 ft^3/s with a chamber inlet dimension of 3 ft (D_1).

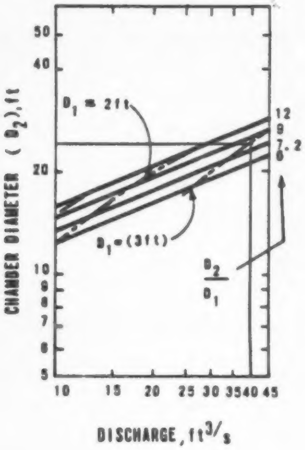


Figure 7-3. Swirl chamber diameters for 90% settleable solids recovery [29].

$D_2 = 24$ ft

   b. From Figure 7-4 (Figure 15 in reference [29]), check the settleable solids removal efficiency for a 24 ft diameter chamber.

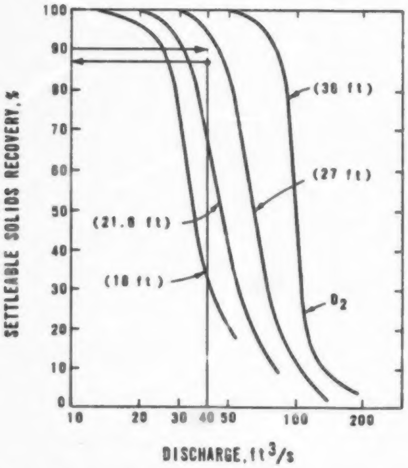


Figure 7-4. Settleable solids recovery for D_1 = 3 ft at H_1/D_2 = 0.25 [29].

Interpolating the recovery curve for D_2 = 24 ft, the swirl efficiency = 87%

c. Adjust the swirl chamber diameter to achieve 90% removal. From Figure 7-4, the D_2 dimension is interpolated from the curves at 90%.

Adjusted $D_2 = 25$ ft

d. Compute height of the swirl chamber (H_1) from relationship $H_1/D_2 = 0.25$.

$H_1 = 0.25 \times 25$ ft
$H_1 = 6.25$ ft

e. Determine the standard design details as shown in Figure 47, using the D_1, D_2, and H_1 values derived above:

$D_3 = 0.67 \times D_2 = 16.7$ ft
$D_4 = 0.56 \times D_2 = 13.9$ ft
$h_1 = 0.50 \times D_1 = 1.5$ ft
$h_2 = 0.33 \times D_1 = 1.0$ ft
$b_1 = D_2 \div 18 = 1.4$ ft
$R_1 = 0.39 \times D_2 = 9.7$ ft
$R_2 = 0.25 \times D_2 = 6.2$ ft
$R_3 = 0.104 \times D_2 = 2.6$ ft
$R_4 = 0.188 \times D_2 = 4.7$ ft
$R_5 = 0.61 \times D_2 = 15.3$ ft

f. Estimate the settleable solids removal over a range of expected flow of 10 to 90 ft^3/s using Figure 7-4.

10 ft^3/s - removal = 100%
40 ft^3/s - removal = 90%
50 ft^3/s - removal = 75%
60 ft^3/s - removal = 47%
70 ft^3/s - removal = 28%
90 ft^3/s - removal = 12%

2. Determine the revised swirl chamber dimensions using a weir height (H_1) equal to the inlet diameter (D_1) of the standard design. The revised swirl concentrator will have the same settleable solids removal efficiency as the standard design. The geometry modification is made utilizing Figure 7-5 (Figure 10 in reference [29]).

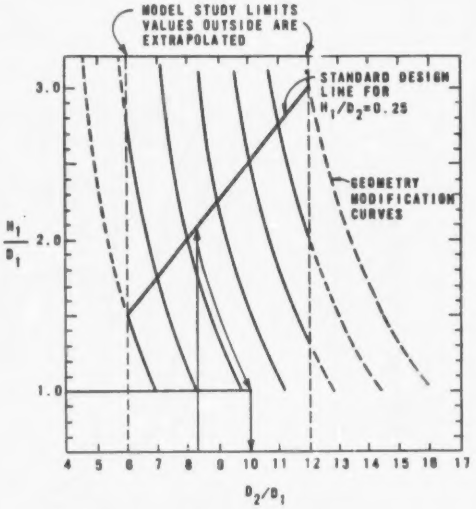


Figure 7-5. Swirl geometry modification curves [29].

a. Compute D_2/D_1 using the standard design values.

$D_2/D_1 = 25/3$
$= 8.33$

b. Enter Figure 7-5 at $D_2/D_1 = 8.33$ and move vertically to the standard design line. A revised D_2/D_1 value is obtained by moving along or parallel to the geometry modification curve to the specified H_1/D_1 value, in this case, $H_1/D_1 = 1.0$; and then down to the revised D_2/D_1 value of approximately = 10.0.

c. Compute the revised chamber diameter (D_2).

$$\begin{aligned} D_2 &= 10.0 \times D_1 \\ &= 30 \text{ ft} \end{aligned}$$

d. The other design dimensions are then recalculated using the new D_1, D_2, and H_1 values.

Comment

In detailing a swirl concentrator/regulator, the designer should choose a swirl inlet dimension approximately the same size as the influent sewer. However, where there is a choice of inlet sizes, the largest inlet size will result in the smallest, most economical structure. It is recommended that swirl designs also include an emergency overflow for flows that exceed peak design capacity. The swirl design curves developed from the model studies are limited by the fact that inlet dimensions of only 1 ft increments are provided for inlets 2 ft and larger; therefore, estimates of swirl size will have to be estimated or interpolated for odd sizes of inlets. The swirl design is also limited by the model study design limits for D_2/D_1 of 6 to 12.

EXAMPLE PROBLEM 7-3: ESTIMATION OF OPERATION AND MAINTENANCE COSTS FOR STORAGE FACILITIES

Develop a normalized operation and maintenance cost relationship such that average annual operation and maintenance costs may be estimated as a function of storage volume.

Specified Conditions

1. Storage volume, capital, and operation and maintenance costs for storage facilities are taken from Table 73.

2. Cost basis: ENR 2000.

Assumptions

1. Annual operation and maintenance costs are adjusted by the total storage capacity and the capital costs to obtain an equal basis of comparison, using the data for several sizes and types of storage facilities.

2. The resulting curves and equations represent an average normalization for any type and size of storage facility and are assumed to include labor, miscellaneous supply costs, and energy costs.

Solution

1. Determine the operation and maintenance cost factor (C_f) for the storage facilities presented in Table 73.

   a. For Akron, Ohio, the C_f is evaluated by dividing the annual operation and maintenance cost by the storage capacity and the capital cost.

$$\begin{aligned} C_f &= \frac{\$2\ 900}{(1.1 \text{ Mgal} \times \$455\ 700)} \\ &= \$0.0058/\text{Mgal}\cdot\$ \text{ capital cost} \end{aligned}$$

b. Operation and maintenance cost factors for the storage facilities are summarized as follows:

Annual operation and maintenance cost, $	Storage volume, Mgal	Capital cost, $	C_f $/Mgal·$
2 900	1.1	455 700	0.0058
51 100	3.9	1 774 000	0.0074
80 000	1.3	6 495 000	0.0095
97 600	1.2	9 488 000	0.0086
100 200	12.4	11 936 000	0.0007
2 700	2.8	744 000	0.0013
6 200	0.36	520 000	0.0331
3 340	0.20	883 000	0.0189
14 400	0.25	320 000	0.1800

c. The cost factors are plotted against storage volume, as shown in Figure 7-6, along with the best fit curve representing the average normalized conditions.

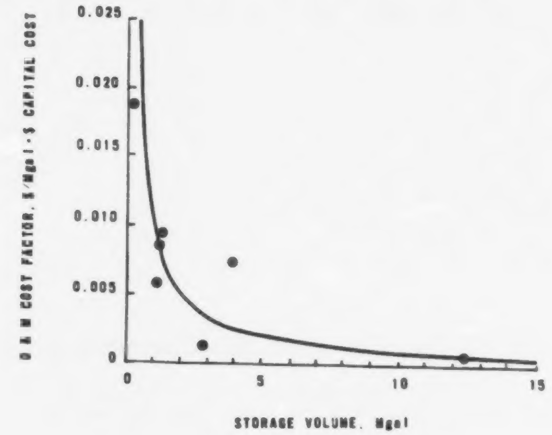


Figure 7-6. Operation and maintenance cost function for storage facilities.

d. The best fit curve has the equation:

$$C_f = 0.0105\ V^{-1.0476} \qquad (7\text{-}1)$$

where C_f = cost factor, $/Mgal·capital cost
V = storage volume, Mgal

the correlation coefficient = 0.86

2. Develop a normalized operation and maintenance cost equation for storage facilities using the best fit curve equation from Figure 7-6.

Annual operation and maintenance costs are found by multiplying the cost factor at the required storage volume by the storage volume and the estimated capital cost:

$$\begin{aligned}\text{Operation and maintenance cost} &= C_f \times V \times C_c \qquad (7\text{-}2)\\ &= (0.0105\ V^{-1.0476})\ V \times C_c\\ &= (0.0105\ V^{-0.0476})\ C_c\end{aligned}$$

where V = storage volume, Mgal
C_c = capital cost, $

3. Compare the results of the operation and maintenance cost equation with estimates obtained from the cost curves developed for stormwater facilities [27]. Capital costs for use in the equation are taken from the storage reservoir capital cost curve for concrete uncovered storage basins, Figure 34, to make an equal basis for comparing the operation and maintenance cost curves [27]. The comparison for storage facilities of 2, 5, 10, and 15 Mgal capacity is summarized as follows:

Storage volume, Mgal	Capital cost, \$ (Figure 34)	Operation and maintenance cost, \$/yr (Equation 7-2)	Operation and maintenance cost, \$/yr[a] (cost curves [27])
2	500 000	5 100	5 880
5	900 000	8 750	7 950
10	1 300 000	12 200	11 300
15	1 700 000	15 700	13 600

---

[a] Includes labor interpolated for 40 events per year at \$10/h.

Comment

The operation and maintenance costs determined by Equation 7-2 provide a means and flexibility for estimating costs on a first-cut basis for both large and small storage facilities of simple or complex design and operation. Operation and maintenance costs based on the complexity of the design or process are controlled by the capital cost of the facility as well as the volume of storage. The operation and maintenance values generated by the equation, using the capital cost values developed in reference [27], compare favorably with those taken from the curves.

EXAMPLE PROBLEM 7-4: STORAGE/TREATMENT OPTIMIZATION

Evaluate the cost of total treatment and total storage and determine the optimum storage/treatment combination for a given design rainfall at a level of treatment costing \$30 000/Mgal·d.

Specified Conditions

1. Drainage area = 1000 acres.
2. Average runoff coefficient = 0.50
3. Capital cost for treatment = \$30 000/Mgal·d
4. Capital cost for storage = \$1.00/gal
5. Operation and maintenance costs for storage taken from Equation 7-2.
6. Operation and maintenance costs for treatment = 0.015 + 0.027 (treatment cost). Developed for reference [93].

Assumptions

1. Assume storage is to be dewatered in 24 hours.
2. The design rainfall rate = 1.2 in./h
3. The peak rainfall is assumed to be 1.5 x design rainfall.
4. The duration of rainfall equals runoff duration.

Solution

1. Determine the treatment capacity and cost to treat the total runoff. The treatment rate will be designed for the peak flow, without storage or flow attenuation.
   a. Peak rainfall = 1.5 x design rainfall
      = 1.5 x 1.2 in.
      = 1.8 in./h
   b. Determine the peak treatment rate (Q).

$$Q = \frac{(1.8 \text{ in./h} \times 0.50 \times 1000 \text{ acres})\ (43\ 560 \text{ ft}^2/\text{acre})\ (7.48 \text{ gal/ft}^3)\ (24 \text{ h/d})}{(12 \text{ in./ft})\ (1.0 \times 10^6 \text{ gal/Mgal})}$$

$$= 586 \text{ Mgal/d}$$

c. Compute the cost of treatment.

Cost = 586 Mgal/d x \$30 000/Mgal·d
= \$17.58 million

2. Determine the cost of storage assuming the stored volume is dewaterd in 24 hours through treatment costing \$30 000/Mgal·d. Using Equations 4-1a and 4-2a:

a. Storage volume $= 0.02715\ KiAt_1 - (Q \div 24)t_2$: Eq.4-1a
$= 0.02715 \times 0.50 \times 1.2 \times 1000 \times 1.0 - (Q \div 24) \times 1.0$
$= 16.29 - (Q \div 24)$
$= 16.29 - (16.29 \div 24)$
= 15.61 Mgal

b. Cost of storage/treatment $= 0.02715\ KiAt_1C_2 + Q\left[C_1 - \frac{C_2t_2}{24}\right]$: Eq. 4-2a

$$= 16.29 \times 1.0 + 16.29\left[0.03 - \frac{1.0 \times 1.0}{24}\right]$$

$= 16.29 - 0.19$
= \$16.10 million

c. Evaluate the cost of storage and treatment individually for this situation.

Storage cost = 15.61 Mgal x \$1.0/gal = \$15.61 million
Treatment cost = 16.29 Mgal/d x \$0.03/gal·d = \$0.49 million

3. Determine the optimum storage/treatment combination using annual capital costs and total annual costs (including operation and maintenance).

a. Compute the storage volume required to reduce the peak treatment rate to the average design treatment rate, using the linear relationship shown in Figure 7-7.

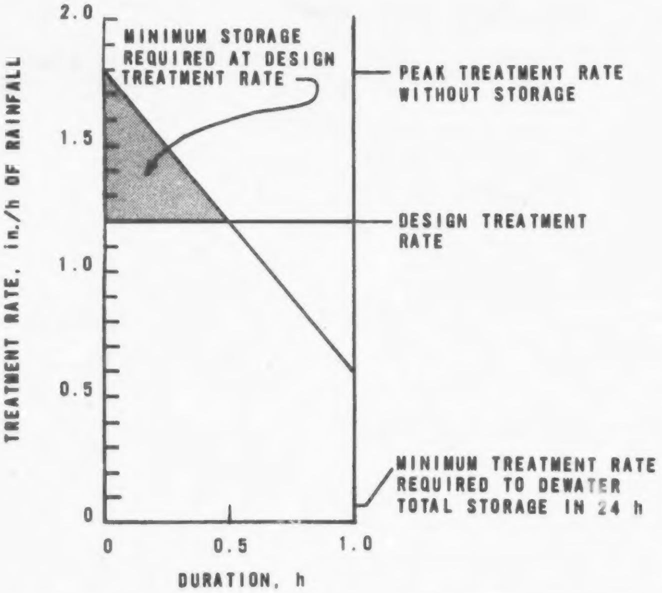


Figure 7-7. Relationship of treatment rate and storage volume for treatment rates greater than 0.6 in./h of rainfall.

The shaded area represents the storage volume required to provide the average design treatment rate of 1.2 in./h of rainfall. Area = 1/2 bh.

$$\text{Storage volume} = \frac{(0.5[1.8\ \text{in./h} - 1.2\ \text{in./h}] \times 0.5\ \text{h})\ (0.50)\ (1000\ \text{acres})\ (43\ 560\ \text{ft}^2/\text{acre})\ (7.48\ \text{gal/ft}^3)}{(12\ \text{in./ft})\ (1.0 \times 10^6\ \text{gal/Mgal})}$$

= 2.04 Mgal

b. Compute the treatment rate at 1.2 in./h of rainfall.

$$\text{Treatment rate} = \frac{(1.2\ \text{in./h} \times 0.50 \times 1000\ \text{acres})\ (43\,560\ \text{ft}^2/\text{acre})\ (7.48\ \text{gal/ft}^3)\ (24\ \text{h/d})}{(12\ \text{in./ft})\ (1.0 \times 10^6\ \text{gal/Mgal})}$$

$$= 391\ \text{Mgal/d}$$

c. Determine the cost of storage and treatment at the design treatment rate.

Storage cost = 2.04 Mgal x \$1.0/gal = \$2.04 million
Treatment cost = 391 Mgal/d x \$0.03/gal·d = \$11.73 million

Total cost = 2.04 + 11.73 = \$13.77 million

d. Compute the storage/treatment costs for other treatment rates.

Note: At treatment rates of less than 0.6 in./h of rainfall, Equations 4-1a and 4-2a may be used. At treatment rates greater than 0.6 in./h of rainfall, the storage volume is computed from Figure 7-7 by multiplying the area of the triangle at the desired treatment rate by the appropriate conversion factors.

Costs of several storage/treatment combinations are as follows:

Rainfall, in./h	Treatment rate, Mgal/d	Storage volume, Mgal	Treatment cost, \$ million	Storage cost, \$ million	Total cost, \$ million
0.05	16.29	15.61	0.49	15.61	16.10
0.1	33	14.92	0.99	14.92	15.91
0.2	65	13.58	1.95	13.58	15.53
0.4	130	10.87	3.90	10.87	14.77
0.5	163	9.50	4.89	9.50	14.39
0.6	195	8.15	5.85	8.15	14.00
0.7	228	6.84	6.84	6.84	13.68
0.8	261	5.66	7.83	5.66	13.49
0.9	293	4.58	8.79	4.58	13.37
1.0	326	3.62	9.78	3.62	13.40
1.1	358	2.77	10.74	2.77	13.51
1.2	391	2.04	11.73	2.04	13.77
1.3	424	1.41	12.72	1.41	14.13
1.4	456	0.91	13.68	0.91	14.59
1.5	489	0.51	14.67	0.51	15.18
1.6	521	0.23	15.63	0.23	15.86
1.7	554	0.06	16.62	0.06	16.68
1.8	586	0	17.58	0	17.58

e. The total capital costs are coverted to amortized capital costs assuming a 20 year life at 7% interest. Compute the annual capital cost at a treatment rate of 16.29 Mgal/d.

Annual cost = \$16.10 million x 0.09349
= \$1.520 million/yr

f. Compute the annual operation and maintenance costs for each storage/treatment combination. The storage and treatment operation and maintenance costs at a treatment rate of 16.29 Mgal/d is computed below:

$$\text{Storage operation and maintenance} = 0.0105 \times 15.61^{-0.0476} \times 15.61 \qquad (7\text{-}2)$$
$$= \$0.144\ \text{million/yr}$$

$$\text{Treatment operation and maintenance} = 0.015 + (0.027 \times 0.49)$$
$$= \$0.028\ \text{million/yr}$$

g. Determine the total annual cost for each storage/treatment combination. The total annual cost for a treatment rate of 16.29 Mgal/d is determined below:

Total annual cost = 1.520 + 0.144 + 0.028
= \$1.692 million/yr

Total annual costs for treatment rates up to 456 Mgal/d are presented in the following and are plotted in Figure 7-8.

Treatment rate, Mgal/d	Amortized capital cost, $ million/yr	Operation and maintenance costs, $ million/yr		Total annual cost, $ million/yr
		Storage	Treatment	
16.29	1.520	0.144	0.028	1.692
33	1.502	0.138	0.042	1.682
65	1.466	0.126	0.068	1.660
130	1.394	0.102	0.120	1.616
163	1.358	0.090	0.147	1.595
195	1.321	0.077	0.173	1.571
228	1.292	0.066	0.200	1.558
261	1.272	0.055	0.226	1.553
293	1.262	0.045	0.252	1.559
326	1.265	0.036	0.279	1.580
358	1.275	0.028	0.305	1.608
391	1.300	0.021	0.332	1.653
424	1.334	0.015	0.358	1.707
456	1.377	0.010	0.384	1.771

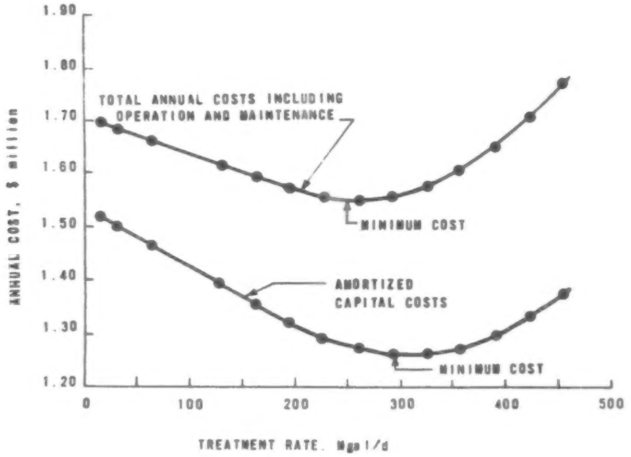


Figure 7-8. Storage/treatment optimization of treatment costing $30 000/Mgal·d.

h. Determine the optimum storage/treatment combination. From Figure 7-8, the optimum solution, using capital costs only, is approximately $12.6 million/yr at a treatment rate of 295 Mgal/d and a storage volume of approximately 4.5 Mgal. With operation and maintenance taken into consideration, the optimum combination is at a treatment rate of 250 Mgal/d with storage at 6.0 Mgal, with a total annual cost of $1.55 million/yr.

Comment

Storage is usually required before treatment of storm and combined sewer overflows to attenuate peak flows and reduce the size of the treatment facility. There is an optimum combination of storage and treatment that produce the least capital cost solution. When operation and maintenance cost is included, the least cost combination shifts to a more storage-intensive solution. Furthermore, as the unit cost of treatment increases, the least cost solution also favors more storage and less treatment. An evaluation of the optimum combinations of storage/treatment at unit treatment costs of $35 000, $40 000, and $45 000/Mgal·d using both amortized capital costs and total annual costs shows that as the unit treatment cost increases, the optimum treatment rate moves toward the minimum treatment rate of 16.3 Mgal/d. The addition of operation and maintenance costs also shifted the optimum rate toward 16.3 Mgal/d as shown below:

Unit treatment cost, $/Mgal·d	Optimum treatment rate, Mgal/d Using amortized capital costs	 Using total annual cost
35 000	260	195
40 000	220	16.3
45 000	16.3	16.3

EXAMPLE PROBLEM 7-5: LAND TREATMENT OF STORMWATER

Determine the land requirements for wetlands, rapid infiltration, and overland flow stormwater land treatment systems. Show the maximum and minimum land requirements based on annual and weekly application rates.

Specified Conditions

1. Drainage area = 1000 acres.
2. Runoff coefficient = 0.50
3. Average annual rainfall = 44 in.
4. Use the design criteria shown in Table 118.

Assumptions

1. The design weekly rainfall equals the total storm rainfall of 1.6 in. as shown in Figure 7-1.
2. The effects of storage or flow attenuation are not considered in determining the land requirements using the weekly rainfall rate.

Solution

1. Determine the annual and the weekly runoff volume from the 1000 acre area.

   a. $$\text{Annual runoff} = \frac{(44\ \text{in./yr})\ (0.50)\ (1000\ \text{acres})\ (43\ 560\ \text{ft}^2/\text{acre})}{12\ \text{in./ft}} = 79.86 \times 10^6\ \text{ft}^3/\text{yr}$$

   b. $$\text{Weekly runoff} = \frac{(1.6\ \text{in./wk})\ (0.50)\ (1000\ \text{acres})\ (43\ 560\ \text{ft}^3/\text{acre})}{12\ \text{in./ft}} = 2.90 \times 10^6\ \text{ft}^3/\text{wk}$$

2. Determine the maximum and minimum land requirements for wetlands treatment, using design criteria from Table 118.

   a. $$\text{Maximum land requirement} = \frac{79.86 \times 10^6\ \text{ft}^3/\text{yr}}{4\ \text{ft/yr}} = 19.97 \times 10^6\ \text{ft}^2$$
   or 458 acres

   b. $$\text{Minimum land requirement} = \frac{(2.9 \times 10^6\ \text{ft}^3/\text{wk})\ (12\ \text{in./ft})}{25\ \text{in./wk}} = 1.39 \times 10^6\ \text{ft}^2$$
   or 32 acres

   c. Compute the annual application rate at the minimum land requirement condition.

   $$\text{Maximum annual application rate} = \frac{79.86 \times 10^6\ \text{ft}^3/\text{yr}}{(32\ \text{acres})\ (43\ 560\ \text{ft}^2/\text{acre})} = 57\ \text{ft/yr}$$

3. **Determine the maximum and minimum land requirements and the maximum annual application rate for a rapid infiltration system.**

a. Maximum land requirement

$$= \frac{79.86 \times 10^6}{20}$$

$$= 3.99 \times 10^6 \text{ ft}^2$$

or 92 acres

b. Minimum land requirement

$$= \frac{(2.9 \times 10^6)\ (12)}{120}$$

$$= 2.5 \times 10^5 \text{ ft}^2$$

$$= 6.7 \text{ acres}$$

c. Maximum annual application rate

$$= \frac{79.86 \times 10^6}{(6.7)\ (43\ 560)}$$

$$= 273 \text{ ft/yr}$$

4. **Determine the maximum and minimum land requirements and the maximum annual application rate for an overland flow system.**

a. Maximum land requirement

$$= \frac{79.86 \times 10^6}{10}$$

$$= 7.99 \times 10^6 \text{ ft}^2$$

or 183 acres

b. Minimum land requirement

$$= \frac{(2.9 \times 10^6)\ (12)}{16}$$

$$= 2.18 \times 10^6 \text{ ft}^2$$

or 50 acres

c. Maximum annual application rate

$$= \frac{79.8^6 \times 10^6}{(50)\ (43\ 560)}$$

$$= 37 \text{ ft/yr}$$

Comment

The ranges of application rates presented in Table 118 were developed for municipal wastewater treatment systems and, therefore, should serve as first-cut guides until more detailed studies using land treatment processes for controlling stormwater are evaluated [78]. These ranges reflect a wide variation in soil types, permeability, slope, climate, and vegetation cover. In this example, the range of annual application rates was narrowed by considering land area requirements based on a design weekly rainfall rate. The land requirements for wetlands range from 3 to 46% of the watershed area. This land, however, would most probably be existing marsh or unusable land areas receiving stormwater discharges directly, or at best an existing marsh operated under a controlled mode of application. Land requirements for rapid infiltration range from 1 to 9%, and for overland flow from 5 to 18% of the watershed area. These land treatment alternatives would require usable or developable land and thus may be limited by land availability and costs.

As with biological treatment systems, overland flow systems were developed for continuous wastewater application to maintain a viable biological mass supported by the grass structure. Because of the intermittent nature of rainfall/runoff, this type of system is reduced to a grass filter for stormwater flows because of the length of time required to develop, stabilize, and sustain a biological mass. Supplemental water may also be required to maintain grass growth during long dry periods. Difficulties may arise with other land treatment methods due to the variability and characteristics of stormwater runoff. Pretreatment may be required for rapid infiltration systems to prevent clogging of the soil by high suspended solids loads.

SECTION 8

SYSTEM APPLICATIONS

As has been indicated in previous sections, there is no one single method that is a panacea to all combined sewer overflow or storm drain discharge problems. The size and complexity of urban runoff management programs are such that there is a need for an integrated approach to their solution. The type of problems associated with any given community is dependent upon a number of variables; as a result, the solution for a community must be developed to fit the needs of that particular urban area. The solution is most often a combination of various best management practices and unit process applications.

Important considerations with respect to development and implementation of an urban runoff management program are the regulatory constraints and public attitudes on pollution and environmental objectives that must be met. Often the constraints and attitudes are subject to change with time. This can result in alteration of the ground rules for engineering assumptions so that programs lacking flexibility may be, or in some cases, have been grossly outdated before implementation can be effected. Thus, the political, economic, and environmental constraints affecting an urban runoff management program must be monitored continuously so that the programs can be updated or modified as necessary.

CASE STUDY DESCRIPTIONS

The presentation of each stormwater management system application is organized into six parts: (1) problem identification, (2) counter-measure philosophy, (3) design description, (4) cost data, (5) performance and maintenance, and (6) ongoing projects. A variety of system applications are described ranging from major urban metropolitan areas to small suburban communities.

Boston, Massachusetts

Combined sewer overflows have contributed to the deterioration of industrial, commercial, and recreational resources of Boston Harbor and the rivers tributary to it [1]. Primary treatment is provided to the intercepted flows at two wastewater treatment plants. However, numerous locations still exist in the Boston Harbor area where, during rainstorms, combined sewage overflows into the receiving waters untreated. These result in bacterial pollution, floating solids, slicks, and sludge deposits.

A wet-weather flow master plan, based largely on preliminary Chicago deep tunnel studies (discussed later in this section), was presented to the City of Boston in 1967 [2]. Four alternatives were studied: (1) complete separation,

(2) chlorination detention tanks, (3) surface holding tanks, and (4) deep tunnels. The deep tunnel alternative was presented because it appeared to offer the best and only feasible method for the complete elimination of overflows. However, following continued review and study of the problems, a demonstration surface detention and chlorination facility was placed into operation in May 1971 at Cambridge, Massachusetts (the Cottage Farm Combined Sewer Detention and Chlorination Station) indicating a viable alternative to the deep tunnel plan.

In 1975, the combined sewer overflow problem was reviewed again in conjunction with the needs for the Boston Harbor-Eastern Massachusetts Metropolitan Area [1]. The major alternatives were (1) sewer separation, (2) overflow diversions via Boston's proposed deep tunnel plan, and (3) intermediate approaches of a decentralized nature. The recommended course of action was to upgrade the two existing treatment plants to secondary treatment and to begin facilities planning for projects identified in the decentralized plan for combined sewer overflow regulation. The decentralized plan would continue present remedial practices and allow piecemeal implementation with immediate opportunitites for solving high priority problem areas. The present plan calls for consolidation of the combined sewer outfalls into several groups, each of which would be connected by conduits to transport overflows to regulation facilities for treatment and discharge.

Treatment would consist of several detention facilities located throughout the area where the flow would be stored or, depending on the magnitude of the storm event, detained prior to discharging the overflow. The flow would be disinfected by introducing chlorine upstream from the tanks. The tanks would be designed to provide 15 minutes detention for the peak design flow. The tanks would include floating scum baffles and screens installed between the scum baffle and the overflow weir to polish the overflow before discharge. The stored flow would be returned to the interceptor to receive secondary treatment at one of the two treatment plants.

According to the Report:

> ...the largest benefits in pollution reduction in decentralized systems will probably come from first flush capture and diversion to the dry weather flow treatment plant and through sedimentation, skimming and disinfection as a result of detaining overflows, while other treatment processes will be employed where such prove to be necessary for further polishing. [1]

The total cost for the various alternatives ranges from \$254 to \$279 million (ENR 2000) excluding projects currently underway (separation in portions of Cambridge and Somerville and construction of the MDC Charles River Chlorination-Detention-Pumping Station Project).

Chicago, Illinois

In 1967, the Metropolitan Sanitary District of Greater Chicago initiated its wastewater facilities planning study with a 10 year cleanup and flood control program. A major study to develop a comprehensive program for the 972 km^2

(375 mi^2) combined sewer area was completed in 1972. The program, presently being implemented, is the Tunnel and Reservoir Plan (TARP). The objectives of the program are:

> ...to minimize the area's pollutant discharges and the flooding caused by overflows of mixed sewage and wastewater....elimination of the need to release polluted river and canal flood waters into Lake Michigan. [3]

This final TARP is a combination of several alternative plans designed to collect urban runoff during all wet-weather conditions except those storms of a magnitude equal to the three most severe storms recorded to date by the National Weather Service.

Four tunnel systems comprise the TARP. Each tunnel system consists of three components: reservoirs, conveyance tunnels, and sewage treatment plants. A total of three reservoirs, 201 km (125 miles) of conveyance tunnels, and four treatment plants are included in the plan. The combined storage capacity of the plan is approximately 167 750 000 m^3 (44 310 Mgal) of which 11 350 000 m^3 (3 000 Mgal) is tunnel capacity. The total storage capacity is equivalent to 17.3 cm (6.8 in.) of runoff from the combined sewer area, with 1.2 cm (0.46 in.) of runoff capacity in the tunnels alone. The tunnels, located 46 to 88 m (150 to 290 ft) below ground level, range in size from 5 to 10.7 m (17 to 35 ft) in diameter. The total planned treatment capacity will be approximately 96.4 m^3/s (2200 Mgal/d) of which 91.2 m^3/s (2150 Mgal/d) is existing. The stormwater treatment rate would be approximately 31.8 m^3/s (725 Mgal/d) or about 0.5 times average dry-weather flow. More than 640 existing overflow points will be eliminated by the TARP systems. The subsystems common to all TARP tunnel systems include drop shafts, collecting structures, and pumping stations. Pumping stations will be constructed underground at the end of all conveyance tunnel routes and adjacent to all storage reservoirs. These stations will be sized to allow a full tunnel to be emptied within 2 to 3 days.

In addition, instream aeration at more than ten locations along the Chicago River and Calumet Sag Channel are planned to allow the Illinois standards for dissolved oxygen concentrations to be met.

The Phase I system (tunnels and pumping stations without reservoirs) is under construction currently. The TARP costs are estimated at $2 553 200 000 (ENR 2000). The breakdown is as follows:

Conveyance tunnels	$ 869 800 000
Instream aeration	14 000 000
Treatment plant upgrading	986 900 000
Reservoirs and flood control	682 500 000
	$2 553 200 000

Additional costs such as sewers, solids disposal, O'Hare Treatment Plant, and non-TARP flood control will raise the total cost to $2 979 400 000. To date, approximately $45 000 000 of tunnel construction has been completed and another $100 000 000 is under construction.

It is projected that the Phase I tunnel system, with overflows at the existing outfalls until the reservoirs are completed, will reduce the number of overflows to the river system to about ten per year. This will result in a 75% reduction in the volume of combined sewage overflowing to the river and a 90% reduction in the combined sewer overflow BOD_5 mass load to the river.

Detroit, Michigan

Detroit is served by a combined sewer system and a primary treatment plant. In May 1966, an agreement between the Detroit Metro Water Department (DMWD) and the Michigan Water Resources Commission required

> ..the City of Detroit to take immediate steps to decrease the frequency, magnitude and pollutional content of all combined sewer overflows from the City's sewer system to the Detroit and Rouge Rivers. [4]

Detroit considered the following alternatives to meet the agreement: (1) systems management utilizing sewer monitoring and remote control of pumping stations and selected regulator gates to affect in-system storage, (2) complete sewer separation, (3) retention basins to capture storm wastewater, and (4) the above in various combinations. After a review of the alternatives, the systems management approach was selected for implementation in a demonstration project [4].

The system developed includes telemeter-connected rain gages, sewer level sensors, overflow detectors, a central computer, a central data logger, and a central operating console for monitoring and controlling pumping stations and selected regulating gates. This system has enabled DMWD to apply such pollution control techniques as storm flow anticipation, first flush interception, selective retention, and selective overflowing.

The in-system storage potential at locations where remote control facilities were installed was 526 500 m^3 (139.1 Mgal). In addition, there is approximately 581 200 m^3 (150 Mgal) of uncontrolled storage in the system.

Upon receiving advance information on storms from remote rain gages, the operator initiates a sewer pumpdown procedure to increase the available in-system storage capacity. This procedure, along with in-system flow routing, has enabled DMWD to contain and treat many intense spot storms entirely, in addition to many scattered citywide rains.

Since the completion of the demonstration project in 1971, DMWD has continued to expand the monitoring project [4]. The change in the system is indicated in Table 122. The supervisory control system has been expanded with the addition of four new control panels in addition to the original three. Remote control facilities including three wastewater pumping stations, four interceptor regulators, three fabridams, two in-system storage gates, one flow routing gate, and one suburban connection have been added. In addition, four suburban retention basins and 11 suburban pumping stations are now displayed.

The DMWD is utilizing sewer system monitoring data to (1) aid in the operation of the system, (2) predict and verify system response to storm events,

(3) establish priorities for overflow abatement projects, and (4) develop computer control algorithms for the various remote control facilities [4]. Additional in-system and offline storage is being investigated.

TABLE 122. COMPONENTS OF THE MONITORING AND REMOTE CONTROL SYSTEM [4]

Item	1971	1975
Rain gages	14	25
Level sensors	118	214
Status sensors	68	110
Pumping stations/pumps	7/39	10/52
Radar remoting	0	1
Regulators	4	10

Cost data for the additions to the monitoring and remote control system were not reported.

Milwaukee, Wisconsin

The older areas of the City of Milwaukee are served almost exclusively by combined sewers, approximately 6240 hectares (15 400 acres). Along the Milwaukee River within the City of Milwaukee are 62 combined sewer outfalls. Most of these outfalls, 52, are concentrated in the last three miles of the river before it discharges into Lake Michigan. A flushing tunnel which carries dilution water from Lake Michigan discharges at the head of the reach where the overflows are concentrated. This tunnel has been used since 1888 to dilute the river water to reduce odors.

A demonstration project completed in 1974 studied the concept of detention tanks for attenuating combined sewer overflows. Two of the objectives were [5]:

1. Characterize the performance of a combined sewer overflow detention tank in reducing the pollutional load to the Milwaukee River caused by rainfall in the test area.

2. Project the impact of combined sewer overflow detention tanks on the quality of water in the Milwaukee River.

A 14 760 m^3 (3.9 Mgal) detention tank (Humboldt Avenue Combined Sewer Overflow Detention Tank) serving a 230 ha (570 acre) area was constructed and tested. During the 12 month test period, the tank reduced the volume, BOD_5, and suspended solids loads from this combined sewer overflow location by 65 to 70%. Studies evaluating detention tank removal efficiencies of BOD_5 and suspended solids indicated that removal due to volumetric retention is much more significant than removals due to sedimentation [5]. Removals due to sedimentation generally increased total removal efficiency by approximately 5% over removals due to volumetric retention alone.

For purposes of demonstrating the cost impact of the problem, an approximate cost estimate was developed for construction of 13 detention tanks to receive flows from all combined sewer overflow points on the Milwaukee River in the city. These tanks would serve an area of 2350 ha (5800 acre). All tanks would be similar to the Humboldt Avenue facility as far as design criteria are concerned. The implementation of such a series of tanks would be expected to reduce the discharge of pollutants from combined sewer overflows by approximately 80% on an annual basis. The total cost for the facilities would be approximately $45 050 000. This includes $28 300 000 for the tanks, $8 150 000 for pumping stations, and $8 600 000 for sewers. These costs do not include land, right-of-way, contingencies, or additional treatment facilities.

At the present time, the city is proceeding with the development of a combined sewer overflow abatement program incorporating both detention facilities and other treatment methods.

Mount Clemens, Michigan

Combined sewer overflows from the City of Mount Clemens polluting the Clinton River led to a "stipulation" from the Michigan Water Resources Commission in 1967. With regard to combined sewer overflows, the stipulation called for the construction of facilities by June 1972. A demonstration treatment facility was designed to provide treatment to the overflows by means of a series of aerated lakelets with intermediate microscreening, disinfection, and high-rate pressure filtration prior to discharge into the Clinton River [6]. The testing and evaluation of this facility was completed in 1973. One of the conclusions reached regarding the demonstration project was:

> The Mount Clemens treatment concept evaluation indicates that it is a feasible and reliable concept....sampling data has demonstrated that the capability of the treatment concept to acceptably renovate combined sewer overflows for fishing and boating and for lawn sprinkling. All water quality parameters, except the toxic and deleterious substances parameter (not studied), were met. [6]

Annual suspended solids and BOD_5 removal efficiencies of about 95% were reported for the demonstration collection and treatment facility.

As a result of the demonstration project findings, the city has developed a citywide project for the abatement of combined sewer overflows. It was recommended that for a 610 ha (1500 acre) portion of the city a combined sewage interceptor be installed to collect the overflows and convey them to a retention basin, the contents of which would be withdrawn at a slow uniform rate for further treatment. For the remaining 240 ha (600 acre) area sewer separation by constructing new collecting sanitary and/or storm sewers was recommended. Construction of the citywide project began in 1974.

The collection and treatment project involves the interception of overflows (5 year storm) from combined sewers and conveying them to the main pumping station at the retention basin site. The flow will then pass through sedimentation-resuspension chambers before discharge to an aerated retention basin. Any excess will overflow into a chlorination basin before discharge to the Clinton River. Wastewater will be withdrawn from the retention basin at a constant 0.18 m^3/s (4 Mgal/d) rate and conveyed to the existing demonstration project site for treatment. (Dry-weather flow is now treated elsewhere as part of the MACOMB County-Detroit Metro Water Department Regional System.) Treatment will include clarification and disinfection; future chemical additions for phosphate removal will occur at this location. The water will then be discharged to three lakelets in series. The initial lakelet will be an aerated "flow-through" treatment unit. Effluent from the final lakelet will be filtered through high-rate pressure sand filters before discharge to the Clinton River. The city has designated the treatment-park site for development as a recreational facility. The final lakelet is expected to be acceptable for recreational use and potential use for watering park landscaping.

The total construction cost for the sewer separation and the collection and treatment facilities was estimated at $15 140 000. The sewer separation portion was $2 160 000. The total project costs (including engineering, legal, fiscal, administrative, and property and easement acquisition) were estimated to be 125% of the construction cost. The treatment facilities are expected to be on-line early in 1977.

Rochester, New York

Within the Rochester Pure Waters District, combined sewer overflows represent a major pollutional load to the Genesee River, the Rochester Embayment of Lake Ontario, and Irondequoit Bay. A study completed in late 1976 developed a master plan outlining the actions necessary to achieve a cost-effective solution to the receiving water quality impairment caused by combined sewer overflows [7, 8, 9].

The study was divided into three parts:

- Monitoring and characterization of combined sewer overflows and the collection of field data necessary to characterize the drainage areas serviced by the sewerage system
- Pilot plant study to evaluate the applicability of alternatives
- Application of mathematical models to evaluate the effect of combined sewer overflows on the receiving waters to evaluate the effectiveness of various abatement alternatives [8]

Three classifications of processes were piloted: (1) solids removal; (2) chemical precipitation to achieve a greater degree of fine solids removal along with phosphorus reduction below the 1 mg/L level; and (3) final polishing and high-rate disinfection to achieve a secondary quality effluent with respect to BOD_5 and bacterial contamination. The processes investigated were flocculation/sedimentation with and without chemical addition, microscreening, grit swirl and primary swirl concentrators connected in series, dual media filtration, carbon adsorption columns, and high-rate disinfection with chlorine and/or chlorine dioxide.

The alternatives investigated included nonstructural alternatives (source control measures and improved sewer system maintenance practices); minimal structural alternatives (improvement of existing dry- or wet-weather storage and treatment facilities); and structural intensive abatement alternatives (new storage and treatment facilities). Mathematical models were applied to evaluate these alternatives. The runoff block of the Storm Water Management Model (SWMM) was used to evaluate the effects of the nonstructural alternatives. Minimal structural alternatives were evaluated using the SWMM transport block. To determine the average annual effect of various abatement measures, the Simplified Stormwater Model was used [8].

The recommended master plan calls for the implementation of interceptor improvements, regulator modifications, blockage of high impacting overflows, addition of control structures, implementation of source control regulations, implementation of an overall control system, construction of wet-weather treatment facilities at the existing Van Lare Treatment Facility (dry-weather flows) site, and inline tunnel storage and conveyance. The cost-effective optimum structural intensive solution based on the 2 year design storm involves a 12.05 m^3/s (275 Mgal/d) wet-weather treatment capacity and a storage capacity of 227 100 m^3 (60 Mgal). The recommended wet-weather treatment facilities are chemically assisted flocculation/sedimentation (1 mg/L polymer and 40 mg/L alum) followed by high-rate disinfection. The estimated costs associated with implementation of this master plan are \$7 140 000 ± 25% for the nonstructural and minimal structural alternatives and \$88 570 000 ± 20% for the structural intensive storage and treatment alternative [7]. These costs do not include drainage relief facilities that are part of the costs reported in Section 2.

The effectiveness of the proposed master plan was reported as follows:

> ...incorporating the nonstructural and minimal structural recommendations is projected to reduce the BOD_5 and TKN (total Kjeldahl nitrogen) annual wet-weather loading to the Genesee River from approximately 363 600 kg/yr (800 000 lbs/yr) and 9 090 kg/yr (20 000 lbs/yr) to 1360 kg/yr (3000 lbs/yr) and 114 kg/yr (250 lbs/yr). This will reduce the average annual potential of dissolved oxygen contraventions of the Genesee River from approximately 10 days/yr to 1 day/yr.
>
> ...The annual CSO (combined sewer overflow) loading of suspended solids to the Genesee River as a result of implementing the Master PLan will be reduced from approximately 1 363 600 kg (3 000 000 pounds) to a value of less than 4545 kg (10 000 pounds). [7]

Rohnert Park, California

The City of Rohnert Park has separate sanitary and storm sewers. However, high wet-weather wastewater flows are encountered in the sanitary sewers during the rainy season (October through April). Approximately 95% of the average annual rainfall occurs during this period. Peak wet-weather flows exceed average dry-weather flows by as much as eight to ten times [10].

A demonstration project, completed in 1973, was undertaken to determine the effect of a surge facility to provide equalized flows to the dry-weather treatment plant. A unique method for maintaining the flow of solids through the basin was tested. One of the objectives of the study was to compare the primary sedimentation tank efficiencies for variable versus uniform flow conditions [10].

The ability of the equalization basin to produce the design uniform flowrate was documented. The basin operated less efficiently than a conventional clarifier for suspended solids and BOD_5 removal due primarily to the variability in the detention time. The BOD_5 removals were quite erratic.

Following completion of the demonstration project Rohnert Park joined in the Laguna Regional Wastewater Treatment Facility. Rohnert Park (including the Town of Cotati and Sonoma State College) is limited to an average dry-weather flow of 0.10 m^3/s (2.3 Mgal/d) and a peak dry-weather flow of 0.18 m^3/s (4.1 Mgal/d) to the regional plant. Peak wet-weather flow at the old, existing plant site is 0.53 m^3/s (12.0 Mgal/d).

The abandoned Rohnert Park treatment plant has been converted to a surge facility for wet-weather flows. The surge facility has a surge basin (old primary sedimentation basin), a storage basin with two days' detention at maximum daily flow, a control building, and a chlorination facility for emergency wet-weather overflow. Most of the components were retained from the abandoned plant. The storage basin is composed of three unlined earthen basins approximately 1.5 m (5 ft) deep with a combined area of 6.9 ha (17 acres). Total storage capacity is 83 300 m^3 (22 Mgal). Flows in excess of 0.18 m^3/s (4.1 Mgal/d) (are diverted to the surge facility for storage. When the flow in the interceptor to the regional plant falls below 0.18 m^3/s (4.1 Mgal/d). flow is released from the surge facility. Construction of the surge facility was completed in 1976.

Construction cost for the surge facility was $943 000. This was composed of $390 000 for pumping station rehabilitation, $273 000 for the diversion structure and chlorination facility, and $280 000 for storage basin earthwork (including regrading and sludge removal from existing oxidation ponds).

Saginaw, Michigan

The problem at Saginaw was typical of most such systems, namely periodic overflows from the combined sewer system. The distribution of the total intercepted flow among the 34 regulators was inequitable with some contributing a disproportionately large percentage. When flows reached 2.5 times the dry-weather flow, the treatment plant capacity, a valve on the interceptor was closed manually and the flow from one half of the interceptor system was pumped untreated to the river. The valve was reopened manually after the storm when personnel were available. This contributed unnecessarily to the amount of wastes discharged through overflows [11]. In 1969, it was recommended that existing intercepting and stormwater pumping facilities be utilized to their optimum in conjunction with five new stormwater holding facilities. The holding facilities were to have a storage capacity of 85 100 m^3 (22.4 Mgal).

In 1972, following application of the Storm Water Management Model (SWMM) to simulate the operation of the sewer system and proposed storage facilities, the plan was revised [12]. The revised plan called for construction of seven storage facilities with a total capacity of 68 800 m^3 (18.2 Mgal). In addition, revisions to existing regulators would add 70 400 m^3 (18.6 Mgal) of in-system storage. The size of the required interceptors was also reduced as a result of the SWMM simulations. The sizing is based on the 1-year storm, 4.8 cm (1.9 in.) of rain.

To date, one of the storage facilities is under construction and one about to go to bid. In each facility, as flow enters the covered structure, floating scum and oil baffles rise with the liquid surface to maximize capture of these materials. Depending on the magnitude of the storm, when the basin is filled, effluent passes through horizontal screens (1.25 cm (0.49 in.) mesh) to capture any floatable and suspended material not captured in the settling bays before overflow to the Saginaw River. Influent to the facility is disinfected with sodium hypochlorite. Stored flow is dewatered into the interceptor following the storm.

The capital costs for the entire system (seven storage facilities, regulator modification, etc.) were estimated at $44 800 000.

The storage facilities are being designed for multiple use. The two facilities designed to date include a multistory parking garage above the storage and treatment basin.

The actual construction cost of the Hancock Street facilities was $5 216 000 [13]. Approximately 80% of this cost is attributable to the storage facility. The remainder is for the parking garage.

The overall performance of the facilities are estimated to be approximately 30% for BOD_5 and 50% for suspended solids removal for the design storm. On an annual basis, approximately 90% of the BOD_5 and 92% of the suspended solids presently discharged to the river would be removed. The basins will completely contain approximately 1.3 cm (0.5 in.) of runoff from the tributary area without overflowing to the river.

San Francisco, California

Overflows occur from San Francisco's combined sewer system when rainfall exceeds 0.05 cm/h (0.02 in./h). When rainfall exceeds this amount much of the city's wastewater, sometime as much as 53 Mm^3/yr (14 000 Mgal/yr) flows untreated into bay and ocean waters at many points around the city.

A wastewater master plan for an improved wastewater treatment system was developed by the Department of Public Works and its consultants between 1969 and 1974. Since 1974, parts of the plan have been changed as a result of further design and planning work. As the city proceeds with its 8-year program, further changes are anticipated.

The master plan contemplates the establishment of two treatment plants: a dry-weather flow facility in the southeastern area of the city (San Francisco Bay side) and a combined dry- and wet-weather flow facility in the southwestern area (Pacific Ocean side). Both plants will ultimately discharge to the ocean via a common ocean outfall system. Phase I of the plan is shown in Figure 60 [14].

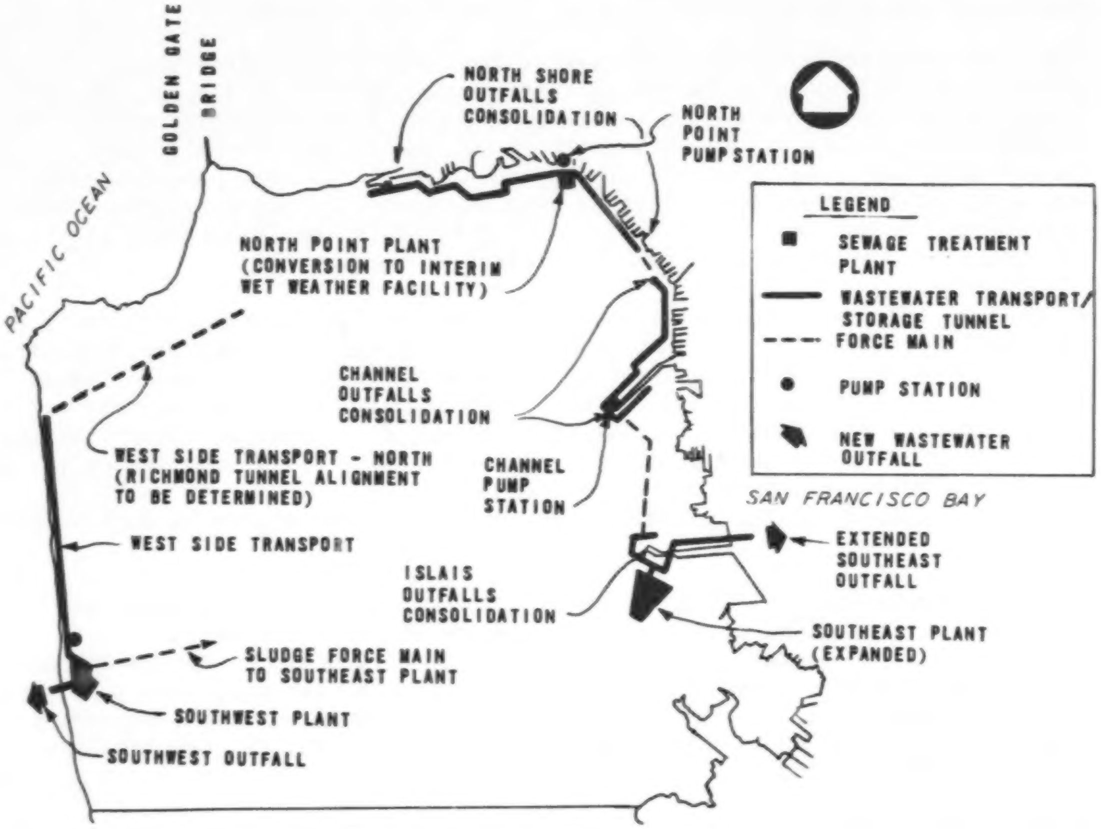

GOLDEN GATE BRIDGE
NORTH SHORE OUTFALLS CONSOLIDATION
NORTH POINT PUMP STATION
PACIFIC OCEAN
LEGEND
■ SEWAGE TREATMENT PLANT
WASTEWATER TRANSPORT/ STORAGE TUNNEL
FORCE MAIN
● PUMP STATION
NEW WASTEWATER OUTFALL
NORTH POINT PLANT (CONVERSION TO INTERIM WET WEATHER FACILITY)
CHANNEL OUTFALLS CONSOLIDATION
WEST SIDE TRANSPORT - NORTH (RICHMOND TUNNEL ALIGNMENT TO BE DETERMINED)
CHANNEL PUMP STATION
SAN FRANCISCO BAY
WEST SIDE TRANSPORT
EXTENDED SOUTHEAST OUTFALL
ISLAIS OUTFALLS CONSOLIDATION
SOUTHEAST PLANT (EXPANDED)
SLUDGE FORCE MAIN TO SOUTHEAST PLANT
SOUTHWEST PLANT
SOUTHWEST OUTFALL

Figure 60. San Francisco wastewater management facilities plan - Phase I.

On the ocean side, the new southwest treatment plant will replace an existing 79 500 m^3/s (21 Mgal/d) plant. The new plant will treat flows for the western half of the city during wet- and dry-weather. A new outfall, presently under design, will be constructed, which will extend out from the southwest plant approximately 6.4 km (4 mi) offshore. Flows treated at the new plant will be discharged to the ocean through this outfall. A large sewage transport/storage tunnel and pumping facilities will be constructed along the west side of the city to the new plant.

On the bay side, the existing 71 900 m^3/s (19 Mgal/J) southeast treatment plant will be expanded to include secondary treatment facilities. The existing capacity will be expanded to 318 000 m^3/s (84 Mgal/d) to treat all dry-weather flows for the east side of the city. The plant will also handle sludge for the entire city. As an interim measure, the existing 260 000 m^3/s (65 Mgal/d) North Point treatment plant (dry-weather flows) will be converted to treat wet-weather flows for the northeastern section of the city. No wet-weather treatment facilities are proposed to handle flows from the southeast section of the city during the initial phase of the program.

The large underground interceptor sewers that make up the North Shore, Channel, and Islais outfalls consolidations and the West Side transport will transport dry-weather flows to the treatment plants or pumping stations, and, during storms, store excess wet-weather flows until they can be treated. These facilities, with the exception of the Channel outfalls consolidation, are expected to reduce the number of untreated combined sewer overflows to an average of one per year. The number of overflows in the Channel outfalls area is expected to be reduced to approximately four per year [14].

As part of the long range plan, a crosstown tunnel and expansion of the southwest treatment plant are proposed [15]. Untreated wet-weather flows from the northeast and southeast districts would be transported to the southwest treatment plant in the crosstown tunnel. This tunnel would be designed for both transport and storage. Treatment of wet- and dry-weather flows from the west side and, during periods of storm runoff, excess flows from the east side would be provided at the expanded southwest treatment plant. Wet-weather treatment capacity at the expanded plant will be approximately 35.0 m^3/s (800 Mgal/d).

The total costs for the first and second stage projects are estimated at $513 300 000 [15]. The estimated cost for the Phase I portion is $308 100 000. At the present time, four of ten contracts for the North Shore and Channel outfalls consolidation projects have been awarded. The total bid costs received for these contracts is $25 700 000 compared to the engineers estimate of $44 750 000. The estimated cost for this entire consolidation project is $86 420 000.

A real time automatic control computer program for inline storage and routing control for the North Shore consolidation project is currently under development. The objectives of this program, when ultimately applied citywide, are: (1) minimization of overflows, (2) priority of the location for discharges when overflows must occur, (3) make maximum use of storage facilities, and (4) make optimal use of all facilities [16].

At present design studies for the ocean outfall, expansion and treatment upgrading along with sludge handling at the southeast plant, facilities planning for the new southwest plant, and the West Side transport and pumping station are underway. A feasibility study of the crosstown tunnel is expected to start shortly.

Seattle, Washington

A comprehensive plan for the collection, treatment, and disposal of wastes from Seattle and other communities within the drainage basin was completed in 1958. Despite improvements brought about by the basinwide construction plan, Seattle itself was still plagued by overflows from the 60-year old combined sewer system. A demonstration project was begun in 1967 to achieve "the ultimate in system storage and control in a combined sewer system through computerized 'total system management'" [17]. This resulted in the development known as the "Computer Augmented Treatment and Disposal System," or CATAD.

The CATAD system is a computer-directed system for maximum utilization of available storage in the trunk and interceptor sewers to reduce or completely eliminate combined sewer overflows. The CATAD system utilizes a computer-based central facility for automatic control of remote regulator and pumping stations. The control center includes a computer, its associated peripheral equipment, an operators console, an interceptor system map display, data loggers, and event printers.

At the same time that the Municipality of Metropolitan Seattle (METRO) was developing the CATAD system, the City of Seattle was proceeding with complete or partial sewer separation projects in several areas of the city. The end result was that the CATAD system serves approximately 5310 ha (13 120 acres) of combined sewers. Of the city's total of 21 060 ha (52 000 acres), the sewer separation area amounted to 7290 ha (18 000 acres).

Remote monitoring and control units were provided to 37 remote pumping and regulator stations. In addition, six remote rain gages are also monitored. The CATAD system can be operated in three different modes: (1) local control, (2) supervisory control, and (3) automatic control. Under local control each station is operated independently by controllers within the station in response to local sensing devices. In the supervisory control mode, stations are operated remotely from the central terminal by the operator via the CATAD computer in response to telemetered data. Stations are operated from the central terminal under program control by the CATAD system computer in the automatic control mode.

Using supervisory control, the volume of overflows was reduced by 35 to 50%. Adding automatic control strategies improved these reductions to over 90% [18]. An optimizing model is being developed that is expected to maintain a performance of at least 80% annual overflow volume reduction. Conclusions reached as a result of the demonstration project include:

> Loading analysis reveals that 80 to 90% of the peak loading has been reduced, and the peak loading has been shifted to a higher rainfall rate which occurs less frequently. Total loading in pounds has been decreased an average of 58% for ammonia; up to 76% for COD.
>
> Rainfall intensity has a considerable effect on overflows. Considering the average rainfall rate of a storm, the total system reduced overflow

volumes by 73.6% in supervisory control, 97.2% in automatic control, and 85.8% under combined advanced control modes.

> Each station tended to show a "fingerprint" effect for sequential overflow data. This fingerprint was generally unique for each station and usually repeated itself for different storm types. The data indicated that the first flush of materials is often diverted to the interceptor in a combined system rather than overflowing to the receiving water.
>
> Overflow priorities were based primarily upon volume reduction. Station by station priority varied considerably depending on which pollution factor was the basis for establishing priority.
>
> During the course of the study, the Duwamish River receiving water has improved dissolved oxygen content by 1 to 2 milligrams per liter. [18].

The success of the application of total systems management concepts is aided by the improved surveillance afforded by the continuous monitoring capability. But the greatest part of the improved performance is due to the ability (under either supervisory or automatic control) to locate portions of the sewer system which can be utilized for storage, thereby allowing overburdened portions of the system to flow more freely [18].

The modifications to the existing combined sewer system included combined sewer separation work by the City of Seattle affecting about 25% of the combined sewers in the CATAD area; modifications to and construction of regulator and pumping stations by the City of Seattle; modification of regulator stations required for CATAD by METRO; and acquisition and interfacing of the telemetry system, controls, and computer for CATAD by METRO. The total cost for the modifications and acquisitions was \$165 650 000. The cost associated with just the CATAD system (regulator station modifications, telemetry system, and control and computer equipment) was \$8 390 000. These costs on a unit area basis were \$5110/ha and \$260/ha (\$12 625/acre and \$640/acre), respectively.

The Woodlands, Texas

A new town, The Woodlands, is under development 56 km (35 mi) north of Houston, Texas. The town will contain all services of a modern city, including facilities for social, recreational, education, commercial, institutional, business, and industrial pursuits. When development began in 1972, the 7200 ha (17 780 acres) was just heavy forest. Development will span 20 years and lead to homes for approximately 150 000 people.

The basic drainage system planned for The Woodlands was designed on the basis of what was termed the "natural drainage" concept. This concept consists of the following principles:

> (a) the existing drainage system in its unimproved state is utilized to the fullest extent possible; (b) where drainage channels need to be constructed, wide, shallow swales lined with existing vegetation are used

> instead of cutting narrow, deep ditches; (c) drainage pipes and other flood control structures are used only where the natural system is inadequate to handle increased urban runoff, such as in high-density urban activity centers; and (d) flow retarding devices such as retention ponds and recharge berms are used where practical to minimize increases in runoff volume and peak flow rates due to development. [19]

It was originally estimated that utilizing the "natural drainage" concept would keep the drainage system costs down to about 50% of that for conventional systems. As part of the initial planning, the impact of the planned urbanization in The Woodlands community was evaluated using the Storm Water Management Model (SWMM). The results were used to develop a program to minimize impact of further development.

To minimize the amount and rate of increased runoff due to urbanization, existing drainage courses are grass covered to slow and reduce runoff through infiltration. Storage reservoirs are used to promote recharge of groundwater and attenuate runoff. Examples of the use of natural drainage features and storage reservoirs are shown in Figure 61. Erosion control measures in construction areas minimize solids loadings in runoff from these areas. The type and amount of fertilizers, pesticides, and herbicides are controlled to minimize pollution of runoff [20].

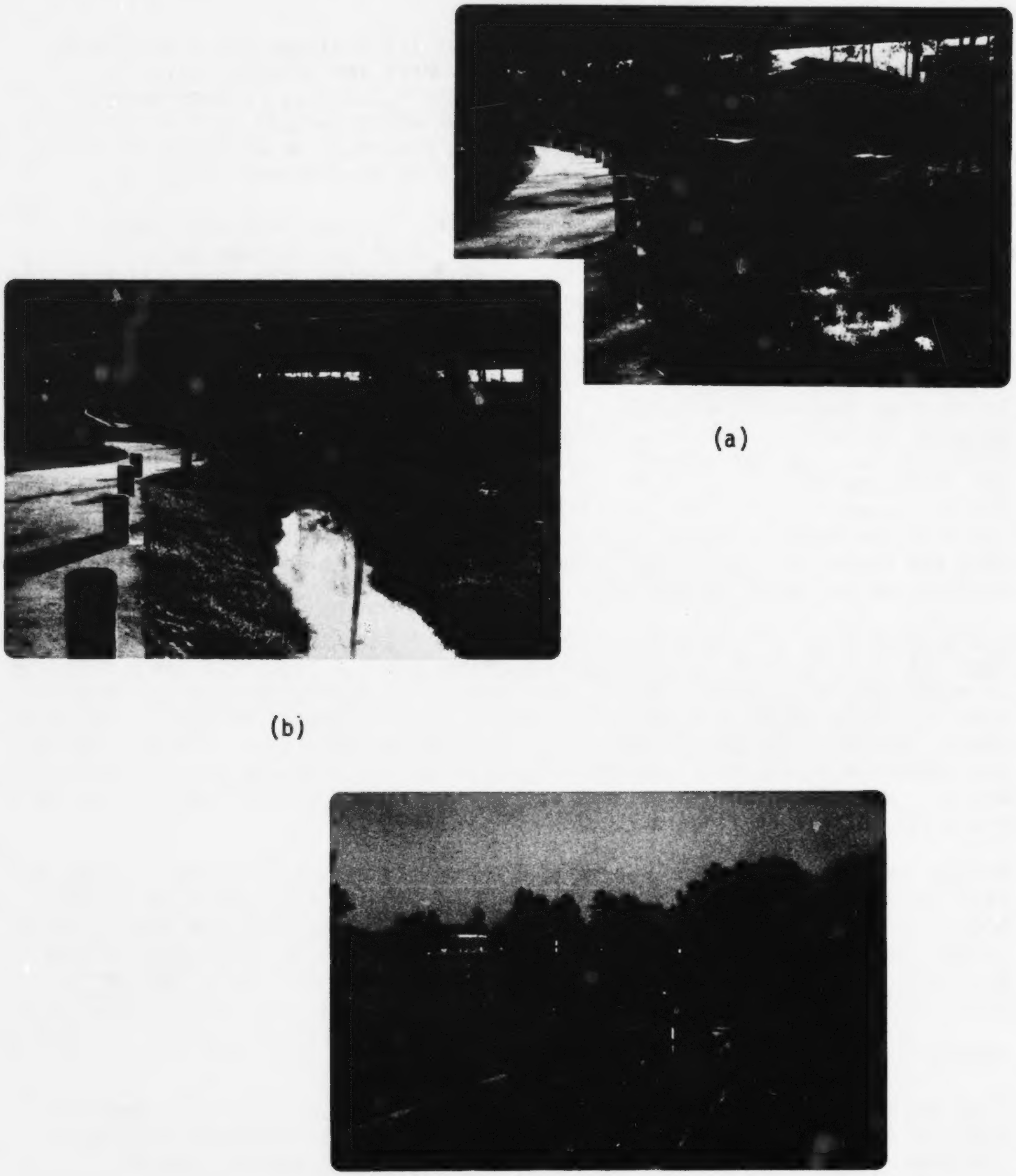
The Woodlands terrain in many places is quite flat. In a recent review it was reported that in such spots, natural drainage has been found to cause flooding of homesites [21]. Also, Houston area officials dislike the natural drainage idea--drainage swales and ditches accumulate debris and silt, and bushes grow there. Removing the debris and bushes is a maintenance cost. These officials feel sewers are less of a problem. The goal is still to use natural drainage wherever practical, but to balance ecology with practical economics since no one wants to live on flooded land.

Part of the original intent was to provide multifamily and cluster housing to keep the developed land to a minimum, thus minimizing the increased runoff from urbanization. However, many Houstonians who can afford new housing want single-family housing [21]. This may result in a smaller percentage of The Woodlands land left in open space than was originally planned. This would most likely increase the amount and rate of runoff.

SUMMARY

From the case studies presented and summarized in Table 123, it is apparent that all use an integrated approach toward solving the stormwater pollution problems. The programs developed by communities with combined sewers generally rely on structural methods to solve the overflow problems. For communities with separate sewers, the stormwater abatement programs incorporate both best management practices and structural solutions. This difference in approaches is probably best explained by comparing the types of communities with combined or separate sewers.

Most of the combined sewers are found in the older, highly urbanized cities. As a result, the more easily implementable and least costly best management



(a)

(b)

(c)

Figure 61. Natural drainage and storage reservoir, The Woodlands, Texas. (a) and (b) Natural drainage swales. (c) Stormwater storage basin used as a recreational reservoir in planned development.

TABLE 123. COMPARISON OF CASE STUDIES IN VARIOUS CITIES

Item	Boston	Chicago	Detroit	Milwaukee	Mt. Clemens	Rochester	Rohnert Park	Saginaw	San Francisco	Seattle	The Woodlands
Year study completed	1975	1972	1974	1974	1973	1976	1973	1972	1974	1973	....
Avg annual rainfall, in.	42.8	33.2	30.9	27.6	27.4	31.5	29.3	28.4	20.8	38.9	45.3
Avg summer rainfall (May-Sep), in.	16.9	17.1	14.5	14.6	12.9	13.4	1.65	14.2	1.0	7.2	20.8
Design storm	1 yr, 6 h	....	....	....	5 yr	2 yr, 2 h	....	1 yr	....	....	....
Type and total sewered area, acres	Combined 17 000	Combined 240 000	Combined 688 000	Combined 5800	Combined 2100	Combined 12 000	Separate 12 800	Combined 10 200	Combined 24 000	Combined 13 120	Separate 17 780
Storage volume, Mgal	21.6-39.3	Tunnels 3000 Total 44 120	Controlled 139.1 Total 293	40.4	Retention basin 33.2 Lakelets 18.9		22	Tanks 18.2 Total 36.8	222.6	17.8	....
Storage volume, in. of runoff	0.05-0.09	0.46 tunnels 6.80 total	0.01	0.26	0.79	0.18	0.06	0.5	0.34	0.05	....
Treatment rate, Mgal/d	....	1100 (stormwater)	....	....	4.0	275 (stormwater)	1.8 maximum	320	800 (stormwater)	....	....
Treatment rate as multiple of DWF (includes DWF)	....	1.5	....	....	NA	3.75	1.8	....	8.0	3.0	....
Available treatment	Primary	Secondary	Secondary for stored flows	Secondary	Secondary + filtration	Chemically assisted primary	Secondary	Primary (overflow) Secondary (stored)	Advanced primary	Secondary	....
Problem	CSOs to Massachusetts Bay and Charles River.	CSOs to Chicago River. Flooding in downtown area.	CSOs to Detroit and Rouge rivers and Lake St. Clair.	CSOs to Milwaukee River.	CSOs to Clinton River.	CSOs to Genesse River and Lake Ontario	Heavily infiltrated sanitary sewers.	CSOs to Saginaw River	CSOs to San Francisco Bay	CSOs to Lake Washington, Elliott Bay, and Duwamish River.	Avoid change in runoff rate and volume. Minimize development impact.

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TABLE 123 (Concluded)

Item	Boston	Chicago	Detroit	Milwaukee	Mt. Clemens	Rochester	Rohnert Park	Saginaw	San Francisco	Seattle	The Woodlands
Solution	Chlorination with a minimum of 15 min detention. Screening and skimming of floatables. Dewater to interceptor.	Deep tunnels for storage and transport. Reservoir storage. Upgrade existing STPs. Instream aeration in Chicago River.	In-system storage. Selective overflowing. Remote control of pumping stations and gates.	Storage/detention with disinfection at 13 locations. Dewater to interceptor.	Sewer separation for 500 acres. Treatment for 1600 acres. Treatment includes storage, sedimentation, disinfection, and filtration.	Inline storage in transport tunnel, flocculation/sedimentation with chemical addition, and high-rate disinfection.	Pumping statio rehabilitation. Convert existing oxidation ponds to storage ponds. DWF to regional STP.	In-line storage and storage/detention tanks with disinfection and effluent screens. Parking garage above tanks.	In-line and off-line storage. Computer control of regulator system. Treatment of stored flow.	Partial separation, in-line storage, and computer control of regulator system.	Utilize natural drainageways. Surface storage. Detention ponds. Minimize Storm sewer construction. Use porous pavements and percolation.
Capital cost, $ x 10⁶ᵃ	254-279	Tunnels 870.0 Total 2553	....	45.0	Separation 2.2 Treatment 13.0 Total 15.2	95.7	0.943	44.8	513.	CATAD System 8.4 Total 165.6	....
Capital cost, $/acreᵃ	14 900 to 16 400	Tunnels 4400 Total 12 600	....	7770	Separation 3600 Treatment 8650 Total 7210	7980	74	4400	21 375	CATAD System 640 Total 12 625	....

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a. Based on ENR Construction Cost Index = 2000.

acre x 0.405 = ha
in. x 2.54 = cm
Mgal x 3785 = m^3
Mgal/d x 0.0438 = m^3/s

practices such as onsite retention, erosion control, use of pervious areas for percolation, and use of natural drainage features to attenuate runoff are difficult, if not impossible, to apply. Thus, reliance on structural methods such as storage and treatment is necessary. Separate sewers may be found in the newer portions of some old cities and in suburban communities. In these areas, best management practices are usually more easily implemented. Incorporating best management practices into the stormwater abatement program generally reduces the need for structural solutions.

It is noteworthy that all of the programs incorporate storage in one form or another. This allows a greater stormwater volume to be treated than just relying on the interceptor capacity to convey stormwater to a treatment plant. In most cases, inline storage is included; even where offline storage is used. This allows the stormwater to be treated using the excess capacity at existing treatment plants or allows the use of smaller new treatment plants.

The unit capital costs for the programs range from $1780/ha to $8660/ha ($4400/acre to $21 375/acre) for communities with combined sewers. There are insufficient data to determine a similar range of costs for communities with separate sewers. Direct comparison of the unit costs for the sewer separation and collection/treatment options for Mount Clemens should not be made since separation is being done in an area that is primarily industrial and open space. The costs for collection and treatment of the combined sewer overflows (in areas where this option was selected) were approximately 30 to 60% of the cost for sewer separation in the same areas.

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APPENDIX

Table A-1. NATIONAL RAINFALL-RUNOFF-QUALITY DATA BANK SUMMARY OF DATA - DECEMBER 1976[a]

Location	Catchment	Area, acre	Drainage system	Number of storms with	
				Quantity	Quality
Broward County, Florida	Residential	47.5	S	32[b]	35[b]
	Commercial	39.0	S	--[a]	14[b]
	Transportation	28.4	S	--[a]	4[b]
San Francisco, California	Baker Street	168	C	4	4
	Mariposa Street	223	C	4	4
	Brotherhood Way	180	C	4	4
	Vincente Street, N.	16	S	1	1
	Vincente Street, S.	21	S	1	1
	Selby Street	3400	C	8	8
	Laguna Street	375	C	2	2
Racine, Wisconsin	Site I	829	C	9	9
Lincoln, Nebraska	39 and Holdrege	79	S	20	20
	63 and Holdrege	85	S	15	15
	78 and A	357	S	14	14
Windsor, Ontario	Labadie Road	29.5	S	22	22
Lancaster, Pennsylvania	Stevens Avenue	134	C	7	7
Seattle, Washington	View Ridge 1	630	S	30	30
	View Ridge 2	105	S	5	5
	South Seattle	27.5	S	31	31
	Southcenter	24	S	30	30
	Lake Hills	150	S	7	7
	Highlands	85	S	4	4
	Central Business District	27.8	C	5	5
Baltimore, Maryland	Northwood	47.4	S	14	--
	Gray Haven	23.3	S	29	--
Chicago, Illinois	Oakdale	12.9	C	21	--
Champaign-Urbana, Illinois	Boneyard Creek	2290	S	28	--
Bucyrus, Ohio	Sewer District No. 8	179	C	10	--
Falls Church, Virginia	Tripps Run	332	S	10	--
Durham, North Carolina	Third Fork	1069	S	15	--
Winston-Salem, North Carolina	Tar Branch	384	S	17	--
Jackson, Mississippi	Crane Creek	285	S	17	--
Wichita, Kansas	Dry Creek	1883	S	8	--
Westbury, New York	Woodoak Drive	14.7	S	10	--
Philadelphia, Pennsylvania	Wingohocking	5326	C	16	--
Los Angeles, California	Echo Park	252	S	18	--

a. See discussion in Section 1.

b. Additional data currently being reduced by USGS.

acres × 0.405 = ha

GLOSSARY

Aerated lagoon--A natural or artificial wastewater treatment lagoon (generally from 4 to 12 feet deep) in which mechanical or diffused-air aeration is used to supplement the oxygen supply.

Biological treatment processes--Means of treatment in which bacterial or biochemical action is intensified to stabilize, oxidize, and nitrify the unstable organic matter present. Trickling filters, activated sludge processes, and lagoons are examples.

BMP--Best Management Practices. Nonstructural and low structurally intensive measures for controlling stormwater pollution by attacking the problem at its source.

BOD--Biochemical Oxygen Demand. The quantity of dissolved oxygen used by microorganisms in the biochemical oxidation of organic matter and oxidizable inorganic matter by aerobic biological action. Generally refers to the standard 5-day BOD test.

Combined sewage--Sewage containing both domestic sewage and surface water or stormwater, with or without industrial wastes. Includes flow in heavily infiltrated sanitary sewer systems as well as combined sewer systems.

Combined sewer--A sewer receiving both intercepted surface runoff and municipal sewage.

Combined sewer overflow--Flow from a combined sewer in excess of the interceptor capacity that is discharged into a receiving water.

COD--Chemical Oxygen Demand. The quantity of oxygen required to oxidize organic matter in the presence of a strong oxidizing agent in an acidic medium.

CSO--Combined Sewer Overflow.

Detention--The slowing, dampening, or attenuating of flows either entering the sewer system or within the sewer system by temporarily holding the water on a surface area, in a storage basin, or within the sewer itself.

Disinfection--The art of killing the larger portion of microorganisms in or on a substance with the probability that all pathogenic bacteria are killed by the agent used.

Domestic sewage--Sewage derived principally from dwellings, business buildings, institutions, and the like. It may or may not contain groundwater.

DPD--A method for measuring chlorine dioxide, hypochlorite, free chlorine, and cloramines using the DPD (N. N. Diethyl-p-phenylenediamine) indicator solution.

Dual treatment--Those processes or facilities designed for operating on both dry- and wet-weather flows.

Dynamic regulator--A semiautomatic or automatic regulator device which may or may not have movable parts that are sensitive to hydraulic conditions at their points of installation and are capable of adjusting themselves to variations in such conditions or of being adjusted by remote control to meet hydraulic conditions at points of installation or at other points in the total combined sewer system.

Equalization--The averaging (or method for averaging) of variations in flow and composition of a liquid.

First flush--The condition, often occurring in storm sewer discharges and combined sewer overflows, in which a disproportionately high pollutional load is carried in the first portion of the discharge or overflow.

F/M--Food to Microorganism Ratio. Calculated as the rate of BOD loading in kg (lbs) per day divided by the kg (lbs) of mixed liquor suspended solids under aeration in the contact tank only.

Infiltrated municipal sewage--That flow in a sanitary sewer resulting from a combination of municipal sewage and excessive volumes of infiltration/inflow resulting from precipitation.

Infiltration--The water entering a sewer system and service connections from the ground, through such means as, but not limited to, defective pipes, pipe joints, connections, or manhole walls. Infiltration does not include, and is distinguished from, inflow.

Infiltration ratio--The ratio of rainfall volume entering the sewers to the total rainfall volume.

Inflow--The water discharged into a sewer system and service connections from such sources as, but not limited to, roof leaders, cellar, yard, and area drains, foundation drains, cooling water discharges, drains from springs and swampy areas, manhole covers, cross connections from storm sewers and combined sewers, catch basins, stormwaters, surface runoff, street wash waters, or drainage. Inflow does not include, and is distinguished from, infiltration.

In-system--Within the physical confines of the sewer pipe network.

Intercepted surface runoff--That portion of surface runoff that enters a sewer, either storm or combined, directly through catchbasins, inlets. etc.

Interceptor--A sewer that receives dry-weather flow from a number of transverse combined sewers and additional predetermined quantities of intercepted surface runoff and conveys such waters to a point for treatment.

Intermittent point source--Any discernible, confined, and discrete conveyance from which pollutants are or may be discharged on a noncontinuous basis.

Municipal sewage--Sewage from a community which may be composed of domestic sewage, industrial wastes, or both.

Nonpoint source--Any unconfined and nondiscrete conveyance from which pollutants are or may be discharged.

Nonsewered urban runoff--That part of the precipitation which runs off the surface of an urban drainage area and reaches a stream or other body of water without passing through a sewer system.

Overflow--(1) The flow discharging from a sewer resulting from combined sewage, storm wastewater, or extraneous flows and normal flows that exceed the sewer capacity. (2) The location at which such flows leave the sewer.

Oxidation pond--A basin (generally 2 to 6 feet deep) used for retention of wastewaters before final disposal, in which biological oxidation of organic matter is effected by natural or artificially accelerated transfer of oxygen to the water from air.

Physical-chemical treatment processes--Means of treatment in which the removal of pollutants is brought about primarily by chemical clarification in conjunction with physical processes. The process string generally includes preliminary treatment, chemical clarification, filtration, carbon adsorption, and disinfection.

Physical treatment operations--Means of treatment in which the application of physical forces predominates. Screening, sedimentation, flotation, and filtration are examples. Physical treatment operations may or may not include chemical additions.

Point source--Any discernible, confined, and discrete conveyance from which pollutants are or may be discharged.

Pollutant--Any harmful or objectionable material in or change in physical characteristic of water or sewage.

Pretreatment--The removal of material such as gross solids, grit, grease, and scum from sewage flows prior to physical, biological, or physical-chemical treatment processes to improve treatability. Pretreatment may include screening, grit removal, skimming, preaeration, and flocculation.

Regulator--A structure which controls the amount of sewage entering an interceptor by storing in a trunk line or diverting some portion of the flow to an outfall.

Retention--The prevention of runoff from entering the sewer system by storing on a surface area or in a storage basin.

Sanitary sewer--A sewer that carries liquid and water-carried wastes from residences, commercial buildings, industrial plants, and institutions, together with relatively low quantities of ground, storm, and surface waters that are not admitted intentionally.

SCS--Soil Conservation Service.

Sewer--A pipe or conduit generally closed, but normally not flowing full, for carrying sewage or other waste liquids.

Sewerage--System of piping, with appurtenances, for collecting and conveying wastewaters from source to discharge.

SG--Specific Gravity.

Static regulator--A regulator device which has no moving parts or has movable parts which are insensitive to hydraulic conditions at the point of installation and which are not capable of adjusting themselves to meet varying flow or level conditions in the regulator-overflow structure.

Storm flow--Overland flow, sewer flow, or receiving stream flow caused totally or partially by surface runoff or snowmelt.

Storm sewer--A sewer that carries intercepted surface runoff, street wash and other wash waters, or drainage, but excludes domestic sewage and industrial wastes.

Storm sewer discharge--Flow from a storm sewer that is discharged into a receiving water.

Stormwater--Water resulting from precipitation which either percolates into the soil, runs off freely from the surface, or is captured by storm sewer, combined sewer, and to a limited degree sanitary sewer facilities.

Surcharge--The flow condition occurring in closed conduits when the hydraulic grade line is above the crown of the sewer.

Surface runoff--Precipitation that falls onto the surfaces of roots, streets, ground, etc., and is not absorbed or retained by that surface, thereby collecting and running off.

Trickling filter--A filter consisting of an artificial bed of coarse material, such as broken stone, clinkers, slate, slats, brush, or plastic materials, over which sewage is distributed or applied in drops, films, or spray from troughs, drippers, moving distributors, or fixed nozzles, and through which it trickles to the underdrains, giving opportunity for the formation of zoogleal slimes which clarify and oxidize the sewage.

Urban runoff--Surface runoff from an urban drainage area that reaches a stream or other body of water or a sewer.

Wastewater--The spent water of a community. See Municipal Sewage and Combined Sewage.

CONVERSION FACTORS
U.S. Customary to SI (Metric)

U.S. customary unit			SI	
Name	**Abbreviation**	**Multiplier**	**Symbol**	**Name**
acre	acre	0.405	ha	hectare
acre-foot	acre-ft	1,233.5	m^3	cubic metre
acre-inch	acre-in.	102.79	m^3	cubic metre
cubic foot	ft^3	28.32	L	litre
		0.0283	m^3	cubic metre
cubic feet per minute	ft^3/min	0.0283	m^3/min	cubic metres per minute
cubic feet per minute per 100 gallons	ft^3/min·100 gal	0.00747	m^3/min·100 L	cubic metres per minute per 100 litres
cubic feet per pound	ft^3/lb	62.4	L/kg	litres per kilogram
cubic feet per second	ft^3/s	28.32	L/s	litres per second
cubic feet per square foot per minute	ft^3/ft^2·min	0.305	m^3/m^2·min	cubic metres per square metre per minute
cubic inch	in.^{3}	16.39	cm^3	cubic centimetre
		0.0164	L	litre
cubic yard	yd^3	0.765	m^3	cubic metre
		764.6	L	litre
degrees Fahrenheit	°F	0.555 (°F-32)	°C	degrees Celsius
feet per minute	ft/min	0.00508	m/s	metres per second
feet per second	ft/s	0.305	m/s	metres per second
foot (feet)	ft	0.305	m	metre(s)
gallon(s)	gal	3.785	L	litre(s)
gallons per acre per day	gal/acre·d	9.353	L/ha·d	litres per hectare per day
gallons per capita per day	gal/capita·d	3.785	L/capita·d	litres per capita per day
gallons per day	gal/d	4.381×10^{-5}	L/s	litres per second
gallons per foot per minute	gal/ft·min	0.207	L/m·s	litres per metre per second
gallons per minute	gal/min	0.0631	L/s	litres per second
gallons per square foot	gal/ft^2	40.743	L/m^2	litres per square metre
gallons per square foot per day	gal/ft^2·d	1.698×10^{-3}	m^3/m^2·h	cubic metres per square metre per hour
		0.283	m^3/ha·min	cubic metres per hectare per minute
gallons per square foot per minute	gal/ft^2·min	2.445	m^3/m^2·h	cubic metres per square metre per hour
		0.679	L/m^2·s	litres per square metre per second
horsepower	hp	0.746	kW	kilowatts
inch(es)	in.	2.54	cm	centimetre
inches per hour	in./h	2.54	cm/h	centimetres per hour
mile	mi	1.609	km	kilometre
million gallons	Mgal	3.785	ML	megalitres (litres x 10^6)
		3785.0	m^3	cubic metres
million gallons per acre	Mgal/acre	8353	m^3/ha	cubic metres per hectare
million gallons per acre per day	Mgal/acre·d	0.039	m^3/m^2·h	cubic metres per square metre per hour
million gallons per day	Mgal/d	43.808	L/s	litres per second
		0.0438	m^3/s	cubic metres per second
million gallons per square mile	Mgal/mi^2	1.461	ML/km^2	megalitres per square kilometre
		1461	m^3/km^2	cubic metres per square kilometre
parts per billion	ppb	1.609	km	kilometre
parts per million	ppm	1.0	mg/L	milligrams per litre
pound(s)	lb	0.454	kg	kilogram(s)
		453.6	g	gram(s)
pounds per acre per day	lb/acre·d	1.121	kg/ha·d	kilograms per hectare per day
pounds per cubic foot	lb/ft^3	16.018	kg/m^3	kilograms per cubic metre
pounds per 1000 cubic feet	lb/1000 ft^3	16.018	g/m^3	grams per cubic metre
		0.016	kg/m^3	kilograms per cubic metre
pounds per mile	lb/mi	0.282	kg/km	kilograms per kilometre
pounds per million gallons	lb/Mgal	0.120	mg/L	milligrams per litre
pounds per square foot	lb/ft^2	4.882×10^{-4}	kg/cm^2	kilograms per square centimetre
		4.882	kg/m^2	kilograms per square metre
pounds per 1000 square feet per day	lb/1000 ft^2·d	4.882×10^{-3}	kg/m^2·d	kilograms per square metre per day
pounds per square inch	lb/in.^{2}	0.0703	kg/cm^2	kilograms per square centimetre
square foot	ft^2	0.0929	m^2	square metre
square inch	in.^{2}	6.452	cm^2	square centimetre
square mile	mi^2	2.590	km^2	square kilometre
		259.0	ha	hectare
square yard	yd^2	0.836	m^2	square metre
standard cubic feet per minute	std ft^3/min	1.699	m^3/h	cubic metres per hour
ton (short)	ton (short)	0.907	Mg (or t)	megagram (metric tonne)
tons per acre	tons/acre	2240	kg/ha	kilograms per hectare
tons per square mile	tons/mi^2	3.503	kg/ha	kilograms per hectare
yard	yd	0.914	m	metre